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Final Report
TASK 1A - VBM Design Guidelines
Chapter 5 - Support Requirements

by

Inco Limited, Thompson Division
Thompson, Manitoba

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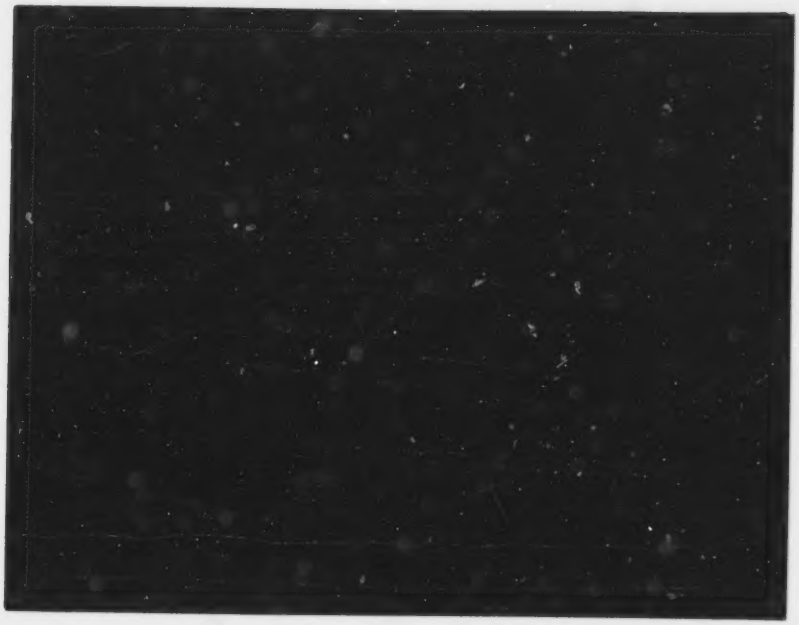

CANADA - MANITOBA

MINERAL DEVELOPMENT AGREEMENT

RESEARCH & DEVELOPMENT IN MANITOBA MINES

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RESEARCH & DEVELOPMENT IN MANITOBA MINES

VERTICAL BLOCK MINING (VBM) DESIGN GUIDELINES

Chapter 5

Support Requirements

This chapter has been prepared as part of the

TASK 1
To Develop Vertical Block Mining Design Guidelines
For A Steeply Dipping Orebody;
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Prepared by

Golder Associates Ltd.
Sub-contracting to
Inco Limited, Thompson Division, Manitoba

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RESEARCH & DEVELOPMENT IN MANITOBA MINES

VERTICAL BLOCK MINING (VBM) DESIGN GUIDELINES

The Vertical Block Mining (VBM) Design Guidelines consists seven chapters, published separately. The seven chapters are:

Summary

1. **Cut and Fill Mining**
2. **Data Collection Systems**
3. **VBM Stope Design**
4. **Drilling and Blasting**
5. **Support Requirements**
6. **VBM Stoping Procedures**

Reference to this chapter should be quoted as follows: "**Vertical Block Mining Design Guidelines Chapter 5 - Support Requirements**"; Canada/Manitoba Mineral Development Sector B - Mining; DSS File No. 14SQ.23440-4-9147-3; CANMET Project No. 4-9147-3; March 1990

EXECUTIVE SUMMARY

Chapter 5: Support Requirements

Tentative guidelines for the selection and design of support in development openings have been developed. These guidelines are based on current industry practice, accepted design procedures and experience gained at Inco's Thompson Mine.

The objective of the support guidelines proposed is to provide a rational basis for selecting support type and density to ameliorate those problems that have the greatest impact on mining, that is back failure, access loss or deterioration, and hangingwall slough. Though primarily oriented towards development excavation support, the guidelines also address the requirements for stope walls.

Any procedure for support design and selection must be largely empirical due to the many variables involved. Five steps can be identified:

1) Characterization of rock mass
2) Prediction of induced stress over life of opening
3) Prediction of likely ground behaviour/failure
4) Support options available
5) Selection

These guidelines therefore provide procedures associated with steps 1 to 4, while the final step, selection, must remain with the user.

SOMMAIRE

Chapitre 5: Soutènements

Des étapes de cheminement pour la sélection et le design d'un soutènement pour les excavations souterraines ont été développées. Ces étapes sont basées sur les pratiques courantes de l'industrie, sur les procédures de design admises et sur l'expérience acquise a la mine Thompson d'Inco.

L'objectif des procédures de soutènement est de fournir une base rationnelle pour choisir le type et le nombre de supports requis pour améliorer les problèmes d'instabilité qui ont une grande influence sur le minage. Les instabilités qui peuvent être rencontrées sont les effondrements de toit, les blocs non-soutenus ou la détérioration de l'éponte supérieure. Bien que l'orientation principale était basée sur le développement des techniques de soutènement des excavations, ces procédures sont aussi destinées pour le support des épontes dans les chantiers.

Toutes les procédures pour le design et le choix du soutènement sont grandement empiriques car plusieurs variables sont impliquées. Cinq étapes peuvent être identifiées:

1. Caractérisation du massif rocheux.
2. Prédiction des contraites induites duarnt la vie de l'excavation.
3. Prédiction du comportement et les risques d'effondrement de terrain.
4. Soutènement disponible.
5. Le choix.

Ces étapes de cheminement donnent des procédures pour les étapes 1 à 4. Ces étapes sont répétées jusqu'à ce que le concepteur décide du meilleur soutènement.

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1.0 INTRODUCTION

This report, part of a larger study of Vertical Block Mining, describes guidelines for the selection and design of support in development openings. These guidelines are based on current industry practice, accepted design procedures and experience gained at Inco's Thompson Mine.

The need for simple guidelines for ground support design in underground mining has become evident during the last decade. This is largely a result of two factors: first, the trend toward high production, bulk mining systems has increased the demands that are placed on ground support; while secondly, there has been several new and innovative support techniques developed. In contrast, there has been little revision to ground support design procedures and practical education of mine operators as to the impact of the new methods. This has led to misuse and non utilization of support techniques, resulting in lost opportunities for more productive mining.

2.0 OBJECTIVES

2.1 General

Support of mine openings is part of the daily operations of all underground mines in Manitoba and throughout Canada. The purpose of support can be simply stated as the need to keep excavations open and safe for their required life span. This can be achieved by the design of relatively stable excavations, and by the installation of internal and external artificial reinforcement.

Costs associated with support, or conversely no support, can be substantial. For example, support may account for 25 per cent of the total cost of developing an access drift. If inadequately designed for actual ground condition, and more importantly, change of ground conditions, costly rehabilitation may become necessary.

Though support, or lack of support, can be a significant cost centre, little design is generally undertaken. Rather, the premise of "what has worked before" is relied upon. Also, little quality assurance is carried out during support installation or afterwards, during the functional life of the opening.

The design of rock support is dependent on a large number of factors including:

- rock quality (strength, deformability, jointing, etc.);
- prevailing stress;
- change in stress due to adjacent mining;
- size of opening;
- function of opening (e.g., drill drift, drawpoint, stope, etc.);
- consequences of rock behaviour;
- properties of support; and
- cost of support and, conversely potential costs of not supporting.

Because of the complex interactions between these parameters, no single design equation is feasible. A considerable amount of judgement is required based on prior experience and the results of a variety of empirical, analytical and numerical models. Good judgement provides the key to a successful pragmatic design.

2.2 Approach

Thompson has a number of ground control problems that must be addressed in order to maintain a safe working environment, minimize dilution and minimize costly rehabilitation. These problems are listed below according to their impact on mining and frequency of occurrence.

Ground control problems ranked according to impact on mining:

1. Access loss
2. Access deterioration
3. Back failure
4. Hangingwall slough
5. Fill wall stability
6. Blasthole closure
7. Pillar stability
8. Draw point stability

Ground control problems ranked according to frequency of occurrence:

1. Hangingwall slough
2. Blasthole closure
3. Access deterioration
4. Fill wall stability
5. Back failure
6. Drawpoint stability
7. Pillar stability
8. Access loss

The objective of the support guidelines proposed in this report is to provide a rational basis for selecting support type and density to ameliorate those problems that have the greatest impact on mining, that is back failure, access loss or deterioration and hangingwall slough. Though primarily oriented towards development excavation support, the guidelines also address the requirements for stope walls; thus the basic divisions are:

A. Footwall Development (Main haulages and travelways, Footwall access drift and ramps, Stope cross-cuts)

B. Ore Development (Overcut/Drill Drift, Undercut/Mucking Drift, Drawpoints)

C. Stope (back, hangingwall, footwall)

As indicated previously, any procedure for support design and selection must be largely empirical due to the many variables involved. Five steps can be identified:

1) Characterization of rock mass
2) Prediction of induced stress over life of opening
3) Prediction of likely ground behaviour/failure
4) Support options available
5) Selection

These guidelines therefore provide procedures associated with steps 1 to 4, while the final step, selection, must remain with the user.

3.0 FACTORS INFLUENCING SUPPORT DESIGN

3.1 General

A particular state of confinement and stress prevails in the ground prior to excavation. This state is changed by the introduction of an excavation. The effect of such a change is to induce stress and strains within the rock surrounding the excavation. Depending on the characteristics of the rock mass, for example, the degree of natural fracturing and the magnitude of stresses involved, support may have to be given to the rock mass in the vicinity of the excavation to ensure stability.

Over the history of underground excavations many types of artificial support have been used. The diversity of support types, coupled with an incomplete understanding of their application, has resulted in many situations where excavations have been either under or over supported. Determination of support requirements for a given area is by no means a straight forward task. The following list presents some of the factors that influence support design.

- State of stress in rock
- Strength of rock
- Rock mass quality
- Ground water
- Size of excavation
- Purpose of excavation
- Proximity of adjacent openings
- Anticipated changes to ground conditions
- Strength of support
- Dynamic effects
- Deformability of support
- Cost and availability of support
- Ease of installation and skill required.
- Mechanization

Determination of support requirements has been performed by such methods as industry experience, historical precedent, empirical methods, analytical analysis and numerical analysis. The complexity of underground excavations places limitations on all these methods. Selection of support type will depend upon the overall compatibility of the support with the ground. For example, are suitable anchor locations present, are adverse water conditions present, do deformation characteristics of the support and rock mass match, and is strength sufficient.

3.2 Definitions

In order to facilitate discussion, the following definitions and comments have been prepared.

a) **Safety or primary support** is the systematic or arbitrary support which is required to be installed in the excavation cycle to ensure the safety of the working place and the stability of the excavation until the permanent support is installed.

b) **Temporary support** is the support which is required to be installed outside of the cycle to maintain the safety and stability of the opening until the permanent support is installed.

c) **Systematic support** is any support which is installed on a regular pattern. It is based on the general characteristics of the ground.

d) **Arbitrary support** is support which is not installed on a regular pattern, but on an individual appraisal of the potential kinematic instability of isolated blocks of ground.

e) **Permanent support** is the support required to provide the excavation with the final finish and to maintain the stability of the excavation on a permanent maintenance free basis. It may or may not incorporate the safety or temporary support as an integral part, depending upon the requirements and specifications of the contract for construction and the methods and supervision of installing such support.

f) **Rock reinforcement** is any support system which consists of combinations of rockbolts, mesh and shotcrete in any quantities, together or separately.

g) **Active support** involves an element of prestress. The amount of prestress is, however, insignificant in itself. It is rather the confining mechanism of active support which mobilizes the inherent shearing strength of the ground to contribute to its own stability. The sooner active support is installed in the relaxational phase after excavation, the more effective the corresponding benefits. Active support exerts a force against the rock mass which is achieved either by pre-tensioning the support or tensioning induced by rock mass dilation.

h) **Passive support** carries the mass of loosened ground, which increases in volume the later the passive support is installed in the relaxational phase after excavation. The amount of passive support required increases the later it is installed in the relaxational phase.

As the purpose of support is to preserve the integrity of the fractured rock surrounding the excavation, active support is generally preferable.

i) **Support pressure:** the ground surrounding an excavation relaxes progressively with time. The relaxational displacements are very small, if the excavation is self supporting, and are increasingly larger as the ground exhibits a reduced capacity to support itself. The ground contributes to a greater or lesser extent to its own stability and requires only to be augmented partially to ensure adequate stability. The greater the relaxation allowed prior to the installation of artificial support the smaller is the extent to which the ground contributes to its own stability and the larger the extent to which artificial support is required.

Support pressure as defined by Barton et al (1974) signifies the pressure which active measures of support apply to the excavation in inhibiting the relaxation. Support pressure as defined by Terzaghi (1946) signifies the equivalent rock load which comes to bear on belatedly installed passive measures of support.

j) **Internal and external support:** internal support forms an integral part of the rock mass, whereas external support is that placed in the excavation space. Examples of the more common support types are divided as follows:

Internal Support:	Mechanical bolts, grouted dowels, resin bolts, split sets, swellex bolts.
External Support:	Shotcrete, steel sets, timber sets, hydraulic props, cribs, backfill.
Combination:	Cable slings, bolts and straps, bolts and mesh.

Internal support tends to better preserve the integrity of the rock mass by minimizing slip and rotation about discontinuities, and increasing shear and tensile strength. As such it is generally preferable to external support. Other advantages of internal support include:

- does not reduce effective size of opening
- less prone to blasting damage
- less prone to equipment damage
- generally smaller than external support
- more versatile than external support
- installation can be mechanized

k) **Initial stiffness:** elements, such as steel arches or packs, which have low effective initial stiffness fail to prevent early deterioration of the rockwall caused by stress-induced creep movements. If they

provide good areal coverage, such elements are best suited for temporary or supplementary support in poor ground conditions such as at stope-crosscut intersections or in rehabilitation operations. Grouted or end-anchored tendons have good initial stiffnesses, and have a correspondingly wider range of application.

l) **Energy absorption capacity:** in conditions where seismic activity is likely, support systems capable of absorbing large amounts of deformational energy are best able to confine rockburst-driven displacements to acceptable and safe levels.

m) **Areal coverage** expresses the percentage of rock wall directly supported by a support system. Systems providing low coverage, such as individual tendons, are ineffective in preventing falls from between the supported areas. The addition of "fabric" cover (lacing, mesh, shotcrete) can increase coverage to 100%. The success of the mesh/lace systems used in the mining industry can be ascribed not only to their good areal coverage and yieldability properties, but also to the fact they provide excellent tensile reinforcement to otherwise brittle shotcrete fabric cover. Such integrated support systems act to bind and stabilize friable rockwall surfaces, and are capable also of absorbing large static or dynamic movements of the rockwall.

n) The **support resistance** (in kN/m^2) of a tunnel support system expresses the product of element yield strength (in kN) movements of the rockwall, and the density at which elements are installed ($elements/m^2$).

o) **Yieldability:** elements, such as end-anchored tendons or grouted bar tendons or gunite used without mesh or fibre reinforcement, which have low yieldability, fail after only modest movements of the rock mass have taken place. Such elements are unstable for environments where long-term static or dynamic stress changes take place.

3.3 State of Stress

The state of stress in the rock influences the mode of failure and the amount of deformation. Where low compressive stresses, or tensile stresses occur, ravelling, wedge and/or block failure (where kinematically feasible) may occur. Bulk deformation of the rock mass is generally low, though individual blocks, if not constrained, may exhibit large movements.

Where high compressive stresses occur, buckling failure, compressive failure (crushing) and the creation of thin fracture planes by tensile splitting may occur. This may lead to a zone of "failed" or loosened rock around the excavation resulting in substantial deformation of the rock mass.

Support selection must consider the mode of failure as well as expected stress conditions likely to be encountered over the life of the excavation. In low stress environments restraint of key blocks is essential. A rigid support that minimizes the movement of individual rock blocks is desirable. The support should be of such strength to support the dead weight of failed material. In high stress environments, failure around the periphery

usually results in a skin of material under low stress conditions. Support should be such that it minimizes failure yet maintains the integrity of the failed rock. The support must yield, rather than fail, under the applied loads.

Though the excavation of a mine opening causes a redistribution of the prevailing field stresses, the redistributed stresses do not necessarily remain constant during the life of the opening. Adjacent mining can induce further stress changes, therefore support requirements may alter over the life of an opening. It is important to realize that these stress changes may also result in changes to the fabric of the rock mass by inducing further failure. Design of support in changing stress environments therefore can be particularly difficult. Installation of additional support may be necessary to avoid the need for costly rehabilitation.

3.4 Support Deformability

The stiffness of a support system is its capacity to deform when subject to stresses applied by the rock mass. Support systems with a high stiffness, or deformation modulus, yield very little before failure. Support systems with low stiffness yield significantly before failure. Stiff systems are generally stronger but prone to more violent failure.

A good support system should restrain relative displacements between individual blocks, but must accommodate the inevitable bulk movement. If too stiff a support system is installed, the energy usually dissipated through deformation may build to a point where it causes failure. If a support system of insufficient stiffness is installed it may provide inadequate support to be of any use. Where high loading rate conditions occur, such as rock bursting, the support system must yield rather than fail.

A certain amount of rock mass deformation is unavoidable. To some degree the amount can be controlled by how soon after excavation support is installed, and how stiff a support system is installed. The stiffness of a support system is dependent upon such factors as type of support, density of support, length of support and bonding strength in the case of rock bolts. Determining the amount of deformation that is acceptable, and in some instances required, is a very important design consideration.

The importance of support system stiffness and timing of installation is illustrated in Figure 5-1. The curve ABCDE represents the ground reaction curve of an advancing excavation; that is a plot of the deformation of the excavation walls versus the support pressure required to maintain stability. Lines 1 to 4 are the support reaction lines. Deformation is theoretically halted at the point where the ground reaction and support reaction curves intersect.

The radial displacement 0-F is that which is unavoidable. At F, support 1, 2 and 3 are installed, they reach equilibrium with the ground at points B, C and D, respectively. Support 1 is too stiff, the resulting stress buildup may result in failure. Support 2 is less stiff thus allowing considerably more deformation. The rock mass continues to carry the major portion of the load without excessively stressing the support. The support acts to retain the rock mass integrity. Support 3 has a lower stiffness yet. It yields considerably before reaching equilibrium at the point where the rock mass has started to loosen. This support is considered too flexible since it is not capable of maintaining equilibrium if any additional load is applied. Support 4 is identical in nature to

support 2 but installed after radial displacement 0-G has occurred; excessive deformation has occurred as well, the point where equilibrium is reached may be beyond the strength of the support.

Approximate values for stiffness of various support systems is given in Table 5-1 while a simple rating system based on stiffness and yieldability proposed by the Chamber of Mines (1988) is given in Tables 5-2 and 5-3. In actual fact, the stiffness of most support systems is not linear over all applied pressures. This is well illustrated by the results of pullout tests presented in Figure 5-2. For many support systems the stiffness can be varied over their effective length. For example, grouted bolts and cable bolts can be de-bonded over a given interval.

3.5 Purpose of Excavation

The intended purpose and life expectancy of a mine opening will influence the design and selection of support. Obviously where men and equipment are exposed, support design must be conservative. Similarly where the opening is expected to function over an extended period of time, support must also be expected to last accordingly. On this basis, it must be possible to classify the various openings based on use and time they have to function.

3.6 Rock Mass Quality

Rock mass fabric and structure, including overall quality, are very significant in determining the type and amount of support required. The more jointed and broken a rock mass, the weaker and more deformable it will be, and hence the more support that will be required around an excavation.

Rock mass quality describes the degree of fracturing that exists, the condition of the fractures and the strength and deformation properties as well as its overall behaviour. A number of classification systems have been developed which try to quantify overall rock mass quality. Of these the Q System (Barton, 1974) and MRMR system (Laubscher, 1984) are the most widely accepted. A number of general statements concerning rock mass quality and support are given below.

- Competent rock should be supported such that key blocks are unable to rotate or loosen. The support should have a capacity greater than the weight of an individual block.

- Weaker blocky rock should be supported such that dilation is inhibited. This requires active support that maintains shear strength and reduces peripheral deformation. In addition, support must prevent rock fragments from falling.

- Laminated or closely stratified rock should be supported such that inter strata shear strength is preserved, and larger stronger beams are formed by tying individual laminate together.

- Very weak rock requires support that inhibits dilation and prevents deterioration by ravelling. Active support coupled with a blanketing support is necessary.

3.7 Ground Water

The presence of flowing ground water can cause significant difficulties to excavation support. Ground water reduces the quality of the rock by introducing a pore pressure and degrading joint surfaces, and may make installation of some support types impossible and cause corrosion in others.

The presence of ground water indicates open fractures. These fractures may exclude the use of grouted anchors. In some instances large quantities of grout have been pumped into the rock mass to reduce permeability and water ingress.

Where the presence of groundwater is anticipated in an excavation, appropriate action should be taken. Action should include identifying water bearing structures, and if possible avoiding them, and planning a suitable support method in advance.

3.8 Dynamic Effects

Dynamic effects include seismic loading from blasting and rockburst activity. Dynamic loading will cause much higher loading rates than those normally encountered. Where such conditions are expected support capable of sustaining high strain rates should be used. Problems such as loosening support anchors and destroying grout bonds commonly occur. Installed support should be thoroughly checked after experiencing dynamic loading.

4.0 ROCK SUPPORT DESIGN

4.1 General

The behaviour of reinforced rock masses is not completely understood. This is due, in part, to the difficulty in characterizing the mechanical properties of a rock mass which is a discontinuous and heterogeneous material and also accounting for the changing load conditions which occur as mining progresses. Current technology cannot accurately determine support requirements. As such any design method should be used as a guide only.

Four methods of design are used: precedent previously set at the mine, empirical methods, analytical methods and numerical methods. These different approaches have been thoroughly discussed and documented elsewhere, Stillborg (1986), U.S. Army Corps. of Engineers (1980), Barton et al, (1974), Laubscher (1984), Hoek and Brown (1980). Only a general review is presented here.

Many different types of support systems are in use throughout the world, however, most are variations and combinations of a small number of themes. For the purposes of presentation, the support systems have been grouped according to various distinguishing features. Descriptions of the principal support systems are given in Appendix 5-A. These descriptions are general, with little emphasis on the minor differences between support types falling into the same group.

4.2 Support Design Based on Industry Experience

4.2.1 General Ground Support

Existing mining operations contain a wealth of information on support systems. In most cases, these systems have evolved through a trial and error process to the point that they are suitable to the needs of the particular mine. Though each mine has its own specific ground conditions and operational constraints, experience can be transferred from one mine to another, provided that due care is taken.

Ground support practices which are currently in use are summarized in tabular form in Appendix 5-B. This data encompasses a broad range of operational experience at mines in Canada, Australia, Sweden and South Africa. These tables are referenced in the following comments.

Ground support at 66 underground mines in Canada is summarized in Table 5B-1. The data covers rockbolting, grouted anchors, screening and shotcreting where applicable. The evolution of standard practices is evident given the similarities in ground support at many mines. The use of mechanical rock bolts with expansion shell anchors is particularly widespread and is a reflection of this system's relatively low cost, ease of installation and perceived effectiveness. Grouted rebar is also quite common and reflects a trend towards more permanent support of mine openings and support of poorer ground.

Recent studies by Choquet (1986, 1988) have resulted in design charts for rock bolting density based on the practice of mines in the Canadian shield and the Q and RMR rock mass classification systems. The results of this work are shown in Table 5B-2 and Figures 5-3 and 5-4. Figure 5-4 has the general range of rock mass qualities and corresponding rock bolt densities found in Noranda mines (shaded area). It is interesting to note the apparent lack of correlation between the pattern bolting design and geomechanical classification at these mines. This is most likely a result of standardized support practices used by individual mines.

Fabjancyk (1982) carried out a review of ground support practices in Australia. Table 5B-3 shows the range of support types used in development, while a number of pertinent comments contained in the review are summarized below:

Tensioned Point Anchor Support

- Slot and Wedge Bolts: Use declining; where poor conditions encountered bolt type replaced with either resin anchor or fully bonded support types.

- Expansion Shell Bolts: Use declining; often used with straps and plates to contain loose. Efficient in good quality rock where crushing does not occur around anchor. Two major problems are anchor slip and standard patterns providing insufficient support for dead weight.

- Resin anchor bolts: Use increasing (together with fully bonded bolts) becoming the standard type of support for weak rock and for development close to open stopes. Used in a large range of environments, including heavily sheared rock.

Fully Bonded Dowels

Use of fully bonded dowels limited due to cost, productivity and required skill. Generally used where high quality long term support required.

Split Set Bolts

Use limited, due to success of existing methods and because of problems experienced; including limited anchorage and corrodibility. Generally use limited to walls.

Massive Support

- Steel and Timber: Use declining, though several areas of specific application such as fault or shear zones and high tonnage drawpoints exist. Design and installation is generally found to be poor resulting in ineffective support.

- Shotcrete: Use generally limited to major excavations due mainly to logistics.

A detailed review on support practices was carried out by Mount Isa Mines, Australia. This review refers only to Mount Isa itself. Pertinent details of the study are summarized in Tables 5B-4 and 5B-5.

Swedish support practice in development openings has been reported by Aston (1987) and is summarized in Table 5B-6. Again, the change from mechanical anchor bolts to fully grouted dowels is evident.

The South African gold mining industry has carried out extensive research into ground control practices. Though ground conditions are significantly different from those exhibited in Manitoba mines, a considerable experience base exists in support of highly stressed rock. This experience has been formalized into an industry guide (CMRO, 1988) in which support selection procedures are developed. Table 5B-7 summarizes the approach recommended in the guide.

4.2.2 Cable Bolts

Potvin (1988) completed a study on cable bolt use in Canadian open stoping mines. The study concluded that 25% of the investigated mines had underdesigned cable bolt support while many also had over designed support. This condition was believed to be due to the use of design methods developed for rock bolting. To date cable bolt support design is not well developed.

In Canadian mines, for stope backs, the average cable bolt spacing was found to be 1.8 m x 1.8 m. For the most part the range of spacing used is 1.4 m x 1.4 m to 3 m x 3 m.

Table 5-4 presents a survey of cable bolt practices worldwide. This information was collected through literature review and personal communications. Bolting densities throughout the world are observed to be

consistent with Canadian practices. However, in both cases a significant amount of scatter exists even for similar situations.

In an effort to create design guidelines based on precedent, Potvin created design charts for cable bolt density and length, Figures 5-5 and 5-6. The charts make use of the rock quality designation (RQD), joint number (Jn) and hydraulic radius (excavation perimeter/area). These parameters are described in Appendix 5-C. Potvin's comments have been paraphrased as follows:

Cable Bolt Density, Figure 5-5

- The first observation is the scatter of the data. This implies that for similar rock mass conditions the intensity of bolting used at different mines varied greatly.

- The horizontal dashed line indicates that with only one exception, the minimum bolting density used in open stope backs is 0.1 cablebolts/m^2.

- In the zone between the vertical dashed line and the y-axis only two cases have been reported stable out of a total of ten. This suggest that cable bolts are not likely to be effective when the relative block size factor (RQD/Jn/hydraulic radius) is smaller than 0.75.

- The band shown indicates the trend of using of higher density of bolting for smaller block size (and smaller RQD/Jn/hydraulic radius). It can be noted that for most cases plotting inside this zone are stable. Consequently, it appears that a conservative design guideline for the density of bolting is to use the centre of the band. This corresponds to the average density of bolting used with success in similar rock mass conditions.

Cable Bolt Length, Figure 5-6

- The data is quite scattered. Once again it is believed that this can be attributed to the lack of guidelines for support design and a trial and error process.

- The use of cable bolts in very large open stope surfaces has had little success. It can be seen that for a hydraulic radius exceeding ten, seven out of nine support systems have collapsed.

- The minimum stope plane dimension in which cable bolts have been installed has a hydraulic radius of approximately three.

- The minimum cable bolt length included in the data base is three meters.

- A rough and conservative guideline can be derived from the plot of cable bolt length and hydraulic radius. This is shown by the line "L" which correspond to a cable design approximately equal to the span of opening. It also constitutes an approximate average of what Canadian open stope operators have been using with success.

It must be recognized that these charts reflect industry experience. They have not been proven in actual use as yet.

4.3 Support Design Based on Empirical Methods

4.3.1 General

Empirical methods of design are based on observations of rock mass and support behaviour in similar situations. Rock mass classification systems, excavation size, and unstable block conditions are used as a basis for these methods. The methods were developed for use in civil excavations where geometry is typically much simpler than in mines; as such, due care must be taken.

Many simplifying assumptions are inherent with the approach. It is generally accepted that the empirical design methods provide a good starting point for support design, provided that sound engineering judgement is used. These assessments are most useful where preliminary design is necessary but only limited geotechnical data is available. The rock mass classification systems themselves provide a convenient way of comparing different rock mass conditions and relating support design experience.

4.3.2 Q System

Barton et al (1974) analyzed extensive data relating rock mass quality to the behaviour and support requirements of underground excavations. In this scheme, the structure of the rock mass, the frictional characteristics of fractures and the stresses surrounding an opening are assessed and combined to give the Tunnelling Quality Index (Q). Details of how to determine the "Q" value are given in Appendix 5-C.

The index has been related to the span of opening that will stand unsupported, as illustrated in Figure 5-7. The relationship makes use of a quantity termed the equivalent dimension (De) of the excavation. This dimension is obtained by dividing the proposed span length by the excavation support ratio (ESR). The excavation support ratio is related to the use for which the excavation is intended and the extent to which some degree of instability is acceptable. Barton's suggested values for ESR are given in Appendix 5-C.

Where the need for support is indicated, Barton (1974) has proposed that the support pressure (P), in MPa, required to maintain the stability of the excavation can be estimated from:

$$P_{roof} = \frac{0.2 * J_n^{1/2} * Q^{-1/3}}{3 * J_r}$$

where J_r is the joint roughness coefficient, J_n the joint number and Q the rock mass rating all these parameters are discussed in detail in Appendix 5-C. The wall support pressure P_{wall} can be estimated from the same equation, by changing Q for Q' as given below:

$Q' = 5\,Q$, if $Q > 10$
$Q' = 2.5\,Q$, if $0.1 < Q < 10$
$Q' = Q$, if $Q < 0.1$

As a general rule, if computed wall support pressure is lower than 0.04 MPa (approximately the support pressure applied by 5/8 inch rockbolts on a spacing of 1.2 x 1.2 m), wall bolting becomes ineffective and can therefore be avoided.

Figure 5-8 presents a chart, developed by Barton (1986), into which support pressure has been incorporated. The chart relates rock quality, 'Q', and equivalent dimension span to support type and density. Barton's data can be found in tabular form in Hoek and Brown (1980), pages 290-296. It permits rapid comparison to Barton's extensive database.

Where the use of rockbolts is indicated, the bolt spacing can be read from the chart or evaluated from (Choquet, 1988):

$$S = \text{square root of } \frac{C \times 10^{-3}}{P}$$

where:

S = spacing of rockbolts or cable-bolts, in m;

C = allowed working load of rockbolts or cable-bolts, generally the yield load of bolts that are adequately anchored, in kN;

P = P_{roof} or P_{wall}.

Recommendations for support length have also been provided by Barton et al (1974), as follows:

Roof: rockbolts $L = 2 + 0.15\,B/ESR$
Walls: rockbolts $L = 2 + 0.15\,H/ESR$

where:

L is the support length, in m;
B is the span, in m;
H is the excavation height, in m; and
ESR is the excavation support radio.

4.3.3 'MRMR' System

Laubscher (1984) introduced the parameter design rock mass strength (DRMS) for the purposes of relating his Mining Rock Mass Rating (MRMR, see Appendix 5-C) to rock support design. The DRMS is the

estimated strength of the in situ rock mass allowing for various factors that may affect it; i.e., weathering, joint orientation, blasting and MRMR value. The calculation is as follows:

$$DRMS = \frac{MRMR - IRS}{100} \times UCS \times weathering \times orientation \times blasting$$

where:

MRMR = rock mass classification rating (prior to adjustment)
IRS = intact rock strength rating in MRMR
UCS = strength (MPa) used to determine IRS

Weathering:
Orientation: are empirical adjustments to account for adverse conditions.
Blasting:

Weathering: Adjustments between 1 and 0.75. The most significant influence of weathering is on the condition of joints, although RQD and IRS are also affected.

Orientation: Adjustment between 1 and 0.7. Accounts for the orientation of discontinuities with respect to excavation orientation. The attitude of the joints, and whether or not bases of the rock blocks are exposed has a significant bearing on the stability of the excavation. Recommended adjustments for blocks with exposed bases in backs are given in Table 5-5. In walls, including pillar faces, the degree of instability is determined by the plunge and condition of the joint set defining the lower side of the block. Recommended adjustments are also shown in Table 5-5.

Blasting: Adjustments between 1 and 0.8. The excavation method significantly affects the condition of the periphery of openings. Recommended adjustments are shown in Table 5-5.

Laubscher has prepared recommended support selection based on the relationship between DRMS and maximum stress, difference between maximum and minimum compressive stress and minimum stress (see Figure 5-9). The charts are divided into five zones as follows:

- Zone I is a stable zone where no support is required.
- Zone II comprises key blocks (potentially unstable blocks) that require support. In certain cases the whole drift might require support - particularly at shallow depths, where attention must be paid to joint condition, weathering and joint orientation.
- Zone III is a zone in which spalling, rock falls, movement on joints and plastic deformation occur with increasing intensity as the mining environment stress increases. Rock reinforcement potential increases as the DRMS increases and plastic deformation is likely in the lower range. The upper limit of this zone is based on the achievement of a restraint of 6 MPa in the supported rock mass. This limit could be extended if higher restraint were achieved or if the rock mass were consolidated by grouting.

- Zone IV represents areas in which significant spalling, unravelling, movement on joints and deformation are expected. Total rock reinforcement is required and special attention must be paid to repair techniques. Ore extraction must be at the maximum rate as support will not provide permanent access.

- Zone V is characterized by regular collapses and caving, and support is not likely to be successful.

Within each zone, support recommendations have been developed. These are presented in Table 5-6.

A number of comments of Laubscher are relevant.

"Stand-up time, Time-dependent and Area-dependent Failure":

Stand-up time is an indication of how long an excavation of specified spans in a certain rock mass will stand without failing and is indicative of the advance that is possible before the initial support must be installed. As the support concept is to maintain the integrity of the surrounding rock mass, a conservative approach should be adopted. Time dependent failure is the failure of an apparently stable rock mass after a certain period which is usually caused by weathering or the gradual deterioration of joint filling or alteration zones. Another factor is dependent on areas where, even in good ground, the joint orientation is such that 'key' blocks will fall out at certain intervals along a drift, thereby leading to a loosening of the rock mass. The joint orientation adjustment caters for this, but does not indicate the probability of the occurrence of key blocks and, therefore, this situation should not be allowed to develop on extraction horizons.

Cosmetic Support:

It has been observed that support has been installed in stable areas without any sound logic, and it can only be assumed that this has been done for cosmetic purposes. The danger with cosmetic support is that when installed in a stable environment, the technique becomes standard, only to provide inadequate support when the need for support is real. A false sense of security is therefore developed, creating a dangerous work environment.

This last comment is particularly pertinent where a standardized approach to support is used. In these situations, design must be directed toward recognizing ground conditions that are not standard and adjusting support accordingly.

4.3.4 Mathews Method

Mathews et al (Golder, 1981) have developed an empirical relationship between depth of mining, rock mass quality and stope span. The rock mass quality is evaluated with a Q factor which accounts for rock mass strength and structure only. This number is then modified to account for in situ stress, orientation of geologic structure and orientation of the design surface (wall, back, or inclined). The resulting stability number (N) is plotted against an excavation size factor to assess stability of surfaces bounding an opening.

The Mathew's Method is described in detail in Chapter 3, VBM Stope Design. The method is well suited to assessment of stability for open stopes to determine whether or not stability will be a problem.

4.3.5 U.S. Corps of Engineers

These recommendations for rock bolt support design are based on many compiled case histories. The method has been used extensively as a guide for minimum reinforcement in civil excavations. Its application to mining is limited. A summary of the design method is however presented in Table 5-7.

4.3.6 Support Design of Farmer and Shelton

Rules of thumb were established by Farmer and Shelton (1980), based on the experience of various practitioners. The rules provide design guidelines for length and spacing of rockbolts for excavations on rock masses housing clean, tight, discontinuity in surfaces and a maximum of three discontinuity sets. The rules are presented in Figure 5-10.

4.4 Support Design Based on Analytical Methods

4.4.1 General

Analytical methods are used for design of support where structurally controlled failure is anticipated. The methods do not account for stress induced failures. These methods of structural stability analysis can be divided into three groups; limited rock block stability, beam theory and arch theory.

4.4.2 Limit Equilibrium Stability Analysis

This method determines support requirements for wedges or blocks that are free to fall or slide under their own weight. The first step in this analysis is determination of the block shape and size, and whether or not kinematic failure is possible. This can be rapidly done with the aid of stereonets (see Hoek and Brown, 1980). The size, shape and direction of sliding of rock blocks or wedges is conveniently determined by the techniques described by Hoek and Brown. This method has been coded for computer application; the program UNWEDGE, is presented in the Appendix 5-D.

If the block is free to fall support requirements must be based on the dead weight of the rock to be supported. The number of bolts required can be determined from the equation:

$$N = \frac{W \times FS}{B}$$

N = number of rock bolts
W = weight of rock to be supported
FS = factor of safety
B = load bearing capacity of rock bolt

Bolt length must be long enough to ensure adequate anchoring in competent rock. Bolt density must reflect block shape, i.e., a higher density over the thicker sections of the block.

If the block slides, rather than falls, the frictional properties of the sliding surface can be included. The frictional resistance to sliding will depend upon the load normal to the surface. Tensioned bolts installed normal to the surface can substantially increase the frictional resistance and shear strength of the rock mass. The number of bolts required can be determined from the following equation:

$$N = \frac{W\,(FS\sin\beta - \cos\beta\,\tan\phi) - cA}{B\,(\cos\alpha\,\tan\phi + FS\,\sin\alpha)}$$

where:

- N = number of rockbolts
- W = weight of wedge including the weight of any rock that is supported by the wedge
- FS = safety factor
- β = dip of the sliding surface
- ϕ = angle of friction of the sliding surface
- c = cohesive strength of the sliding surface
- A = base area of the sliding surface
- B = load bearing capacity of the bolt
- α = angle between the plunge of the bolt and the normal to the sliding surface

A knowledge of joint properties is required for this analysis. Values can be determined by testing or estimated from reference to other published results. In both cases conservatism should be emphasized. Bolt length must be long enough to ensure adequate anchoring in competent rock. Bolt density must reflect block or wedge shape.

4.4.3 Beam Theory

The beam theory is applicable to the roof where horizontal lamination occurs, and to side walls where vertical lamination occurs. Two analysis are possible; one assumes a competent material exists beyond the laminated material, the other assumes no competent material.

If a competent material exists beyond the laminated material. The support concept is simply that of supporting the dead weight of the material. The bolting pattern is determined from the following equation:

$$c \times s = \frac{B}{FS \times h \times \rho}$$

where:

- FS = safety of factor
- s = bolt spacing, perpendicular to the axis of excavation

c = bolt spacing, along the axis of excavation
h = thickness of unstable layer of rock
ρ = rock density
B = load bearing capacity of bolt

Bolt length must be long enough to anchor into the competent rock beyond the laminated material. Bolts along the excavation abutments should be inclined slightly to account for the presence of shear stresses from the laminated material.

Where the laminated material continues to depth, tensioned rock bolts can be used to cause the individual lamination planes to interact. This reduces the slenderness ratio and effectively increases the stability of the roof or wall. Panek (1964) formulated a design method for horizontally-bedded roofs in weak rock, however the theory is complicated and requires detailed knowledge of conditions, making its application very difficult.

Where vertical joints occur in the roof, behaviour is determined by the lateral thrusts generated by deflection under gravity of the rock blocks or voussoirs.

Studies by Sterling (1980) indicated that roof behaviour can be characterized as a voussoir beam with the following behaviour:

a) a voussoir beam behaves elastically (i.e., the lateral thrust - vertical deflection plot is linear and reversible) over the range of its satisfactory performance, the upper limit of which approaches the peak transverse load capacity;

b) for a voussoir beam with low span/thickness ratio, the most likely failure mode is shear at the abutment;

c) for a roof with high span/thickness ratio, roof span stability is limited by the possibility of buckling of the beam, with no significant spalling of central or abutment voussoirs;

d) a roof with low rock material strength or moderate span/thickness ratio may fail by crushing or spalling of central or abutment voussoirs.

A two-dimensional plane strain and three-dimensional plate analysis method for determining stability of voussoir arches was developed by Beer and Meek (1984). This method has been coded for computer application; details are presented in the Appendix 5-E.

Should analysis indicate a voussoir beam is not stable, its thickness can be increased by installing tensioned rock bolts. Bolt spacing should be determined by joint density and dead weight considerations.

4.4.4 Arch Theory

A more generalized version of the voussoir beam is arch theory. The formation of natural arches above underground excavations is observed in most mines. The arches are the result of the stress redistribution about the opening. The location of the stable arch beyond the excavation is dependent upon the rock mass properties and excavation span. In order to maintain the stability of the natural arch, the loose rock between the excavation boundary and natural arch boundary must be stabilized. Stabilization is best achieved by installing bolts that anchor above the natural arch boundary. Alternatively tensioned grouted bolts may be used to create a voussoir beam below the natural arch.

A number of models have been developed for determining rockbolt length and density for the arch model, i.e., Pender et al (1963), Rabcawicz (1955) and Bischoff and Smart (1975). However, the many uncertainties in these models made their application of dubious value.

Bolt lengths formulae developed from the above mentioned models and used in practice, are given below.

Length of bolt (m) = 1.40 + 0.184 x width of opening (m)
(Derived from hard-rock practice in Scandinavia; IFF (1979)

Length of bolt (m) = 1.8 + 0.013 x [Width of opening (m)]^2
(Pender et al (1963), derived from experience on Snowy Mountains project)

Length of bolt (m) = 0.3 x width of opening (m)
(Rabcewicz (1955), derived from experience with the New Austrian Tunnelling Method.)

Length of bolt (m) = 0.5 x width of opening (m)
(Based on experience of Mount Isa Mine)

The bolt spacing to accompany these formulae is generally:

Spacing = 0.5 x length of bolt. Greater spacing is insufficient for bolt interaction.

4.4.5 Support Interaction

Support interaction refers to the complex interactive relationship between the properties and behaviour of the installed support and those of the rock mass surrounding the excavation. A general discussion of this topic was presented in Section 3.4 Support Deformability. Greater details are presented in the literature by Hoek and Brown (1980), Ladanyi, (1974) and Daemen (1975, 1977).

Because of the large number of variables, rock-support interaction is a very theoretical problem. To date no satisfactory analysis exist. In order to gain an appreciation of the interaction Hoek and Brown (1980) developed an analysis method based upon Ladanyi (1974) and the M and S rock strength criterion. This analysis

has been coded for computer use under the name TUNNEL. The code and user instructions are given in Appendix 5-F.

4.5 Support Design Based on Numerical Methods

Numerical methods to predict stresses around excavations have advanced significantly over the past few years. A number of techniques are in use, the most common being finite element, boundary element, distinct element and finite difference. The various methods are described in detail in Brady and Brown (1984). Rock bolts and discontinuities can be represented in these models by special elements. These elements behave in a theoretical manner, as do all the elements in the models. In the real world, the variable nature of the geology, and timing and quality of support installations make it impossible to predict support loads and stresses with a high degree of accuracy.

An important advantage of numerical models is the ability to do comparative studies of different excavation and support designs. These studies can be done at relatively low cost in the pre-production stage. Analysis can be made of a number of variables to determine the influence on design and thus arrive at a theoretical optimum. The results of these models are very dependent on the accuracy of the input parameters. Models should be calibrated by reference to monitored in situ conditions.

It is foreseen that numerical methods will continue to be advanced and refined. The use of such models is likely to become an ever more valuable tool in the design of rock reinforcement.

4.6 Economic Aspects of Design

Economics play a role in support design that is generally neglected. Though materials may be costed out by the mine, for example the purchase cost of a Swellex bolt versus the purchase cost of a Split Set bolt, rarely are indirect costs of support, or more importantly lack of support, considered.

This role of design has been investigated by Krauland (1984), who illustrated the economic impact of support qualitatively as follows:

"Increasing the amount of support installed during excavation increases the support effect. This results in decreased operation disturbances due to stability problems and in decreased costs for these. The optimum support, from an economic point of view, is one that results in minimum cost for the sum of costs for rock support and costs for operation disturbance as shown schematically in Figure 5-11."

The cost of support can therefore be detailed as:

Cost for support installed at face;

Direct costs	-	support materials
	-	material losses and damages
	-	transportation

- borehole or other preparation
- warehousing
- installation

Indirect costs
- fixed costs for stope, mine

Cost for disturbance:

Direct costs
- complementary support and related costs
- cleanup
- loss of equipment
- additional development

Indirect costs
- fixed costs for stope, mine
- relocation of production
- production losses

Therefore, the most economic support is not governed by direct cost considerations only, but also the economic risks associated with disruption of the operation, as indicated in Figure 5-11.

Though the economic effects of support and disturbance can be described, it is a difficult concept to apply in practice. Support costs are easily obtained; however, the question of whether support should be designed for the entire life span of the opening or rehabilitation carried out, together with the associated costs, is more difficult to resolve. Obviously in any active development opening where mines are exposed, support must be designed such that it fully addresses safety requirements. Any economic considerations can therefore only address any change of support over and above safety requirements. A number of Canadian operations have undertaken economic assessments of support in lateral development drifts and have found it beneficial to install permanent support, as the development advances, rather than re-condition at some later stage.

Work by Bawden et al (1981) illustrates the economic consequences of grade dilution caused by improper stope design and insufficient ground support. A simple rate of return was calculated for a fictitious high-grade orebody at varying dilutions. Results are summarized as follows:

Dilution:	0	10%	20%	30%	40%
Rate of Return:	25%	19%	14%	6%	-2%

Mining rate = 327,000 tonnes/year (360,000 tons/yr)
Grade = 20% Zn
Tonnage = 2,268,000 tonnes (2,500,000 tons)

Dilution in the order of 30% has produced a negligible rate of return for this particular mine. The example illustrates the economic value of adequate ground support in stopes and should provide argument to mine operators seeking reasons to justify the cost of additional ground support.

In terms of direct costs for ground support, an economic comparison has been carried out based on experience at Canadian mines (Choquet, 1987 and Inco, 1989). Costs are quoted relative to a mechanical bolt, and include hardware purchase, drilling, installation and all labor involved. Bolt length is 2.4 m (8 ft).

Support Type	Relative Cost
Mechanical bolt	100
Resin-grouted rebar	232
Cement grouted rebar	128
Swellex bolt	237
Split set bolt	220
Cable bolt (2.4 m)	286
Shotcrete (for coverage of 1.2 m x 1.2 m area)	118

5.0 DESIGN OF UNDERGROUND SUPPORT AT THOMPSON MINE

5.1 General Introduction

Hangingwall failures are the most common type of ground control problem at Thompson Mine, and are significant sources of dilution. Back failures, though less frequent, have more severe consequences. Statistics from ground control studies have indicated hangingwall failure in at least 31 per cent of VBM stopes mined between 1981 and 1987 (126 stopes in database). Visual estimations of overbreak typically range up to 3.6 m (12 ft), with higher values recorded. Typical examples of both hangingwall and back failures in VBM stopes are shown in Figure 5-12.

Hangingwall failure has been correlated with a number of parameters in efforts to determine its contributing factor, and the strongest correlation appears to be with the strength and rock quality of the various geologic units. When schist was present in the hangingwall, failure occurred 34 per cent of the time at an average overbreak of 2.1 m (7 ft). Other rock types resulted in less overbreak when they occurred in the hangingwall. The reasons for this can best be identified in the following table which categorizes rock type by strength and rock quality.

Rock Type	Strength (MPa)	Mean RQD	Mean Q value
Hangingwall Quartzite	140	70	8
Hangingwall Gneiss	150	100	11
Hangingwall Skarn	150	80	9
Hangingwall Schist	70	65	4
Sulphide Ore	70	55	5
Footwall Schist	70	30	1
Footwall Quartzite	200	50	7

As can be seen above, strength and rock quality of both schist and sulphide units are lower than for other rock types. Design of ground support must therefore consider these parameters.

Standard practices currently in use for ground support at Thompson are 2.4 m (8 ft) mechanical bolts (expansion shell anchors) on a 1.2 m x 1.2 m pattern with 102 mm (4 inch) No. 6 and No. 9 gauge screen for all underground development. In addition, the following support systems are used:

Rockbolting:
- 1.8 m (6 ft) Split Sets on 1.2 m x 1.2 m pattern
- 4.9 m (16 ft) mechanical bolts in ore extraction drifts on 3 m (10 ft) spacing
- cable slings used in walls with very slabby ground

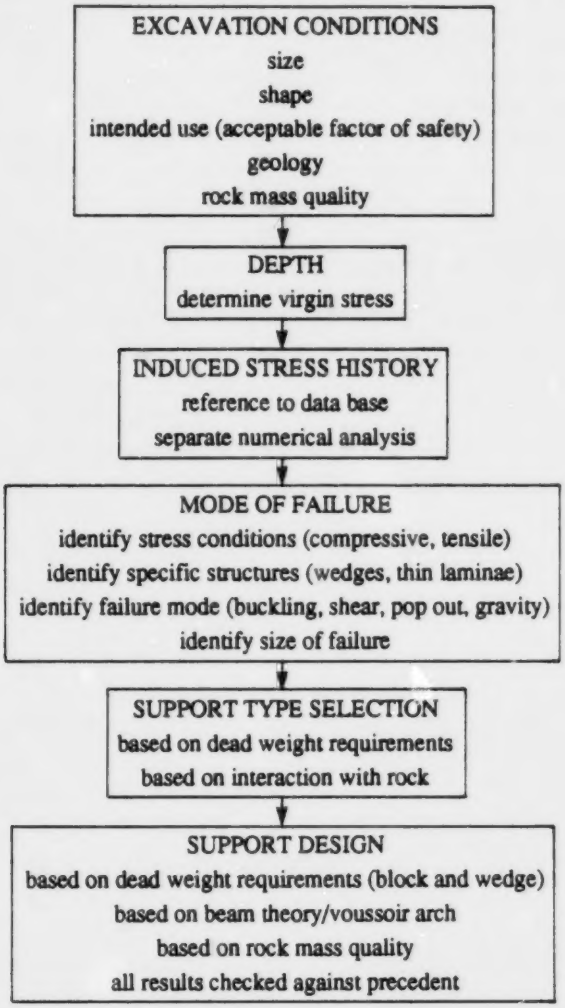
Grouted Anchors:
- 6.1 m (20 ft) cable bolts in drill drift sills on 2.4 m (8 ft) row spacing, 5 per row
- 12.2 m (40 ft) cable bolts in some places

Shotcrete:
- 50 mm (2 inch) thick in drill drift sills and waste drifts with poor ground

The maximum span for development openings in ore is currently set at 6.1 m (20 ft). In thicker ore zones, multiple drill drifts are generally developed rather than silling out wider than 6.1 m and using extra ground support. However, the potential exists to increase this maximum span through incorporation of more extensive support.

The proposed guidelines for support design at Thompson Mine follow a linear path with input from various data bases and computational methods. The following is a flow chart of this proposed path.

FLOW CHART FOR UNDERGROUND SUPPORT DESIGN



Rock mass quality is determined through the empirical methods described previously. Numerical methods are used to estimate induced stress around mine openings. Analytical methods are used to determine the actual support requirements based on the most likely failure modes. Industry and local mine site experience is generally referenced to see what has and has not worked in similar conditions.

Determination of the input values is discussed in the following sections.

5.2 Determination of Input Parameters

5.2.1 Excavation Conditions

Excavation conditions includes the size and shape, geology, rock mass quality and type, i.e. virgin or remnant mining. The purpose of the excavation should also be decided upon for determining an appropriate factor of safety. If after design is completed, it proves to be impractical, then excavation conditions may have to be changed (i.e. size, shape).

5.2.2 Virgin Stress

The stresses surrounding a mine opening greatly influence rock mass behaviour, the stability of the opening and hence support requirements. The magnitude and direction of stresses around an opening are a function of the pre-mining virgin stresses, opening geometry, adjacent mining, and rock mass properties.

In-situ stress measurements have been carried out at Thompson Mine (Miles and Herget, 1979 and SRK, 1984). The results obtained are summarized in Figures 5-13 and 5-14. The major principal stress is observed to be approximately at right angles to the strike of the 1-A, 1-B and 1-C orebodies and dip to the east at between 10 and 15 degrees from horizontal; the minor principal stress is near vertical and coincident with the dip of the regional foliation. From observations made underground and elsewhere in Manitoba it would appear that the major principal stress is coincident with the Mackenzie dyke trend. Though no measurements have been carried out in the Nose, 2-Shear or 3-Shear orebodies the above observations imply that the regional major principal stress is likely at an angle to the trend of these orebodies.

The charts given in Figures 5-13 and 5-14 can therefore be used to estimate the magnitude of virgin stresses within the Thompson Mine. Once an estimate of virgin stress is established, the next step is to assess how this stress state is modified by the creation of mining openings.

5.2.3 Induced Stress

In order to gain a better understanding of induced stresses around development drifts at the Thompson Mine, a simple model was run using Golder Associates NFOLD program. Two scenarios were evaluated: mining a second VBM stope directly above the first (virgin mining), Figure 5-15; and mining the remnant pillar separating two stopes (remnant mining), Figure 5-16. The modeling was performed at mining depths of 300, 460 and 610 m. Thickness of stopes was held at 6.1 m, model elements were 3 m wide. The models were run in forced elastic mode with no consideration of failures. Figures 5-17 and 5-18 present typical stress plots for 50 per cent completion of virgin and remnant mining. These examples are from modelling at 610 m (2000 ft) depth.

The NFOLD runs were used to generate design charts which permit estimation of induced stress in stope backs as extraction increases. These charts are presented in Figures 5-19 and 5-20. Stresses are normalized (induced/virgin) in these graphs for easier comparison. Some general observations can be made with regards to these graphs.

- In virgin mining, the stresses in the back, at the end of mining, are greatest in the centre of the stope and lower towards the sides.

- In virgin and remnant mining stress increases in the elements parallel to the advancing stope face. Stress increases occur in the remnant sill when mining is approximately 60 m away. Stress increases in the back of virgin stopes when mining is 30 to 45 m away.

- High stresses are developed in the remnant sill as it is mined out.

- For a given situation, stress magnitude is proportional to depth.

There are two possible approaches to establishing maximum induced stresses and stress histories at Thompson Mine. Generalized mining scenarios and the above stress data base; or, each mining situation can be modelled separately.

Where mining conditions are close to the generalized models, approximate stresses can be quickly determined. It is necessary that rock types, excavation dimensions, and proximity to other existing or planned excavations be reasonably similar in order to use the generalized stress histories. Where the planned mining conditions do not resemble those in the stress data base, the situation should be modelled independently. Independent modelling is generally preferable as it results in more accurate predictions.

It is extremely important that the stress histories about excavations be investigated. Support based on final stress values only may be insufficient. High stresses generated during the excavation history may cause rock failure or in the extreme, support failure. Where rock failure occurs the stresses will be reduced and the overall rock mass quality decreased. Stress transfer results from the inability of the now fractured rock to carry load. The stresses are transferred further into the rock mass where they are carried by more competent (non failed) rock. The failed rock mass quality is dependent upon what the peak stress was, the extent of new fractures, and what the final stress is. Estimating the final rock mass quality is very difficult due to the number of influential parameters. Prediction using numerical models is possible. However, they must be carefully calibrated by back analysis. Support design must consider changing stress and rock mass quality conditions.

5.2.4 Mode of Failure

Mode of failure must be investigated throughout the entire stress history of the excavation, including induced stresses created by planned future openings. Failure can be either due to high or low stress regimes. It must be noted that throughout its life an excavation may be subject to both compressive and tensile stresses. Furthermore, a high stress regime may cause failure which is manageable. However, a change to a low stress regime may result in failure. The reverse may also occur.

Under a high stress regime, failure can be by buckling, compressive failure, shearing or block pop out. Under a low stress regime failure can be tensile in nature or gravity related. Specific structures must be identified to assist in determination of mode of failure for support design.

The most common type of failure observed has been that of the hangingwall and is believed to be a result of shearing along pre-existing discontinuity surfaces parallel to the excavation periphery (see Figure 5-21). Where the laminations are closely spaced the potential for buckling failure occurs. If the length of the span approaches 1/15 of the thickness of the laminations, support against buckling failure must be considered. Where discontinuities do not exist, the shearing will be through intact rock. Both these situations create wedges and blocks. If high compressive stresses prevail about the blocks they may be squeezed or "popped out" of the surrounding rock mass. If a low compressive or tensile stress regime prevails, failure will be gravity induced.

This fact has implications for the choice of support system. For large deformation type failures such as buckling of walls, a support system which maintains its strength over larger deformations is preferable. For example, cable slings with friction anchors have worked very well for support of buckling failures in walls. For support of gravity type failures such as wedges in stope backs, a more rigid higher strength support system is preferable. Lower deformability systems would include cable bolts and grouted rebar.

A steeply dipping schistosity will often result in preferential fracturing in walls due to the anisotropy of rock strength, as shown in Figure 5-22. The walls have a lower compressive strength than the back due to the orientation of schistosity relative to the loading direction, which is tangential to the rock surface at the opening. Progressive deterioration of walls can undercut the back, as shown in Figure 5-23, thereby creating the potential for back failures. The importance of adequate wall support is obvious under these conditions. This type of failure has often been observed immediately after the final deck blast, where excessive charge weight severely damages the hangingwall beneath the back.

5.3 Support Selection

Where gravity type failures are expected the selected support type must be capable of holding the dead weight of the failed material. Other considerations that need to be addressed include corrosion resistance and suitable support anchoring. Cable bolts and grouted rebar offer considerable advantage over mechanical bolts in both these areas. In the case were support is installed to knit the rock mass together, account must be taken of both the support reaction and ground reaction curves. Factors to also consider include timing of installation and deformability of support system. A support system which satisfies all design criteria can be selected from the available systems described in Appendix 5-A.

5.4 Support Design

After identifying the mode of support and suitable support system, detailed design of support sizing and density can commence.

Where gravity type failures occur, block or wedge analysis should be followed as described in Section 4.4. Where horizontal laminations occur the beam or voussoir analysis described in Section 4.4 should be used.

In the more general situations support will be based upon a quality classification of the rock. Experience suggests the 'Q' method as the most suitable for use at Thompson. Details are given in Section 4.3.2 and Appendix 5-C. Experience has also shown that support will depend on the ratio of maximum boundary stress in the opening to the compressive strength of intact rock.

This fact has been observed at other mines, and data has been tabulated into a design chart by Hoek (1980). The experience at Thompson has been incorporated into this chart, and the results are presented in Figure 5-24. This design chart permits selection of ground support based on rock mass quality, intact rock strength and induced boundary stress. Potential users are warned that this chart should only be used for preliminary assessment of the ground support of proposed excavations.

5.5 Support Design Example

A VBM block is to be developed at a depth of 500 m. The orebody is 12 m wide and dips at 70°. In order to minimize fan drilling it is required to develop the overcut drill drift as wide as possible. The block will be developed to a strike length of 25 m and a height of 50 m.

A nearby drill drift, driven in ore, is selected as representative of the ground conditions which will likely be encountered during development of the new stope. Geotechnical parameters for determination of rock mass quality are obtained using the procedures outlined in Appendix 5-C, and are summarized as follows:

Item	Description	Value
UCS of ore	R4, requires more than one blow to fracture	70 MPa
RQD	Mean joint spacing = 0.1 m	55%
Joint sets	3 joint sets	9
Joint Roughness	Smooth, undulating	2
Joint alteration	Unaltered joint walls with surface staining	1
Joint water	Dry with minor surface staining	1
Stress reduction factor (SRF)	Medium stress	1

Hence, the Rock Mass Rating (Q) is calculated as:

$$Q = \frac{55}{9} \times \frac{2}{1} \times \frac{1}{1} = 12$$

Since virgin stress has not been measured, it is estimated based on previous measurements. Using Figure 5-13 and a depth of 500 m, the vertical stress is estimated as 17 MPa. Using Figure 5-14, the K-ratio of average horizontal stress to vertical stress is estimated as 1.4. Therefore:

σ_V = 17 MPa (vertical stress)
σ_H = 24 MPa (horizontal stress, assumed equal in all directions)

Induced stress in the stope back is estimated using Figure 5-19, which summarizes NFOLD modelling for virgin mining conditions. The modelling was done on 10 VBM blocks over a strike length of 150 m. Individual block dimensions were 15 m length and 60 m height. Our block is 25 m long and 50 m high, or roughly 1 1/2 of the modelled VBM blocks (by area). Average normalized back stress from the chart is 1.9. Therefore:

σ_I = 45 MPa (horizontal stress in back)

Ground support for the stope back can now be selected from the design chart in Figure 5-24. The ratio of maximum compressive boundary stress to compressive rock strength is 45/70 = 0.64. Plotting this ratio against a rock mass quality of 12 yields the following support requirements:

- Presupport with grouted cable bolts.
- Pattern bolting with screen and straps.

Based on previous experience at Thompson, pattern bolting can be accomplished with 2.5 m mechanical bolts on a 1.2 m x 1.2 m pattern, installed with screen and strap.

Since the planned excavation span exceeds 4.5 m, Figure 5-6 will be used to determine cable bolt length. The hydraulic radius of the stope back is defined as the ratio of area to perimeter, and is calculated as follows:

Hydraulic radius = (12 m) (25 m) / 74 m = 4

From Figure 5-6, cable bolt lengths of 8 m have been used successfully at these spans. Cable bolt spacings generally range between 1.4 and 3 m, and a reasonable value for our condition can be estimated from Figure 5-5. This requires a value for (RQD/Jn)/Hydraulic radius, which is calculated as 1.5. From Figure 5-5, a cable bolt density of 0.27 cable bolts/m^2 corresponds to the average density used with success elsewhere in similar rock mass conditions. This is equivalent to 1 cable bolt per 3.6 m^2, or a spacing of 1.9 m (say 2 m). Therefore, cable bolt support for the stope back should use the following criteria:

Cable bolt length:	8 m
Cable bolt spacing:	2 m

It should be emphasized that this is only a preliminary assessment of support requirements. Ground conditions should be assessed as the overcut drift is developed, with particular attention to potential failures caused by adverse geologic structure.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The enclosed guidelines illustrate practical applications for determination of ground support requirements in underground mine openings. These guidelines are based on the collective experience gained at Thompson and other mining operations throughout the world.

In terms of VBM stoping, they are slanted towards support of development openings and stope backs. Support for stope walls is not specifically referenced because walls should be supported by blasted ore in the stope during shrinkage draw. Stability of stope walls is covered in more detail in Chapter 3, VBM Stope Design.

The potential exists to incorporate this design philosophy and experience into a computer-controlled expert system. A system such as this would have to be amenable to continual updating as more experience is acquired.

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TABLE 5-1: SUPPORT STIFFNESS VALUES

Type of Support	Stiffness
Split Set	2,000 to 3,000 kN/m
Mechanically anchored (5/8 in bolt)	3,500 to 5,250 kN/m
Mechanically anchored (3/4 in bolt)	5,250 to 8,500 kN/m
Resin-grouted (3/4 in bolt in a 1 1/4 in hole)	7,800 kN/m
Swellex (in a 1 1/4 hole))	8,500 to 12,000 kN/m
Swellex (in a 1 1/2 hole)	22,000 to 26,000 kN/m
Cement-grouted (3/4 in bolt in a 1 1/4 in hole)	26,000 to 35,000 kN/m
Resin-grouted (3/4 in a 1 in hole)	70,000 kN/m
Shotcrete	20,700 MPa

Reference: Choquet, 1987.

TABLE 5.2 SUPPORT ELEMENT AND SYSTEM CHARACTERISTICS

ELEMENT TYPE	INITIAL STIFFNESS	YIELD-ABILITY	ENERGY ABSORPTN	AREAL COVERAGE	AREA OF APPLICATION
Miscellaneous:					
Steel Arch	Poor	Fair	Poor	Very Good	Temporary support (Rehabilitation etc.).
Split Set	Good	Fair	Poor-Fair	Poor	Primary support only.
End-anchored:					
Rock Stud	Very Good	Poor	Poor	Poor	Mainly primary suport, good-ave. ground.
Cable Anchor	Good	Fair	Fair	Poor-Fair	Tertiary or post-grouted, poor ground.
Fully-grouted:					
Smooth Bar	Very Good	Fair	Fair	Poor	Ave-poor ground, stress increases expected.
Ripple Bar	Excellent	Poor	Poor	Poor	Good ground, no seismicity / high stresses.
Steel Rope	Good	Fair	Fair-Good	Poor-Fair	Poor ground, deep fracturing.
Yield Tendon	Very Good	good - exc	good - exc	Poor	High stresses, seismicity expected.
Fabric:					
Lace	---	Very Good	Good	Good	High streses and / or seismicity expected.
Wire Mesh	---	Very Good	Fair	Very Good	Fragmented ground, low stress changes . (unless in conj, with mesh / lace)
Gunite / Shotcrete	Very Good	Fair	Fair	excellent	
Systems:					
Smooth Bar + Mesh / Lace	Very Good	Very Good	Goood	Very Good	Poor ground , large stress changes and / or seismicity.
Yield Tendon + Mesh / Lace + Gunite	Excellent	Excellent	Excellent	Excellent	

REFERENCE: South African Chamber of Mines, 1988

TABLE 5-3

SUPPORT SYSTEM RATING

RATING SYSTEM	1 Poor	2 Fair	3 Good	4 Very Good	5 Excellent
Early Stiffness (kN/m2/mm)	<0,75	,75-1,5	1,5-2,5	2,5-5	>5
Yieldability (mm)	<25	25-75	75-250	250-500	>500
Rockburst Support Resistance R (kN/m2)	<75	75-150	150-250	250-500	>500
For Tunnels: Energy (kJ/m2)	<2	2-5	5-15	15-25-	>25
Areal Coverage (%)	<3	3-10	10-30	30-50	>50

REFERENCE: South African Chamber of Mines, 1988

TABLE 5-4: SUMMARY OF CABLE BOLT PRACTICES AT SELECTED MINES

	MINE	APPLICATION	BOLT LENGTH	PATTERN	HOLE SIZE	GROUT	BOLT SIZE/TYPE	YIELD LOAD	OTHER
1	Brunswick Mining and Smelting Brunswick #12	Cut & Fill 4.9m lifts	17.7m	1.8m x 1.8m (upholes)	57mm(2 1/4")	high early cement(HE) expansive additive water/cement = 0.4	2 x 16mm (5/8") strand	53 tonne	6mm (1/4") I.D. breather tube
2	Falconbridge Strathcona	Cut & Fill 3m lifts, 3m overlap areas with RQD<50%	17m	1.8m x 1.8m	54mm(2.1")	portland cement-norm water/cement = 0.4 cable buttons used	1 x 16mm strand	27 tonne	6mm breather tube 12.5mm (1/2") grout tube
3	INCO Thompson	Cut & Fill	6.1m, 12.2m	1.2m x 1.2m(1 cable) 2.4m x 2.4m(2 cables)	54mm	portland cement-norm water/cement = 0.45	1 or 2 x 16mm strand	27 or 54 tonne	9.5mm(3/8") breather with Mono pump 12.5mm breather with Speidel pump
4	Con Mine Yellowknife	Cut & Fill 3m lifts + 4.6m H/W	10m, 23m	2.2m x 2.4m	57mm	portland cement-norm water/cement = 0.33	2 x 16mm strand	58 tonne	6mm breather tube
5	New Broken Hill Australia	Cut & Fill 2.4m lifts	20m	2m x 2m	57 mm	high early cement bleeding additive water/cement = 0.51	2 x 16mm strand	53 tonne	3mm breather tube
6	Homestake S. Dakota	Open Stope H/W support	15m	3m x 3m	57 mm	portland cement-norm water/cement = 0.5	1 x 16mm strand	29 tonne	buttons used
		Cut & Fill	18.3m	1.8m x 1.8m	64 mm	portland cement-norm water/cement = 0.5	2 x 16mm strand	58 tonne	12.5mm breather tube buttons used
7	LynnGold MacLellan	Open Stope H/W & F/W	6m	2.4m spacing (2 holes) every 20m sub-level	54 mm	portland cement-norm water/cement = 0.45	2 x 16mm strand	58 tonne	holes drilled from sub-levels at wall contact
8	Placer Dome Campbell	Open Stope H/W support	15m	2.4m spacing (5 holes)	51 mm	portland cement-norm/HE water/cement = 0.4	1 x 16mm strand	29 tonne	holes drilled from sub-levels at wall contact
		Cut & Fill 3m lifts	15m	2.4m x 2.4m	51 mm	portland cement-norm/HE water/cement = 0.4	1 x 16mm strand	29 tonne	support dead weight equivalent to 0.2 m exposed width(F/W - H/W

Reference: Golder Associates

TABLE 5-4: SUMMARY OF CABLE BOLT PRACTICES AT SELECTED MINES
(Cont'd)

	MINE	APPLICATION	BOLT LENGTH	PATTERN	HOLE SIZE	GROUT	BOLT SIZE/TYPE	YIELD LOAD	OTHER
9	West Coast Mines Tasmania	Open Stope H/W support	20m	1.8m x 1.8m (perpendicular to H/W)		portland cement-norm water/cement = 0.5	7 x 7mm wire	45 tonne	holes drilled from sub-levels breather tube 12.5mm, 19m grout tube buttons used
10	Outokumpu Oy Finland	Open Stope H/W support	50m to 15m	1 bolt per 25 sq. meters to to 1 bolt per 50 sq. meters	41mm	portland cement-norm water/cement = 0.5, 0.45	2 x 15mm strand	50 tonne	Fan shaped from F/W and H/W access with spacing 3m to 10m
					64mm	portland cement-norm water/cement = 0.5,0.3	1 x 15 mm strand	25 tonne	Fan shaped from F/W and H/W access with spacing 3m to 10m
11	Mamberget Mines Sweden	Open Stope F/W support	25m to 15m	2.5m along strike (1 hole) every sub-level (20m vertical)		portland cement-norm water/cement = 0.5, 0.45	1 x 36 mm strand	90 tonne	drilled at F/W contact from sub-level RQD of F/W 20 to 60%
12	Mount Isa Mines Ltd. Australia	Open Stope H/W support	9-12m	2m spacing (3 holes) every sub-level (30m vertical)	57-70 mm	portland cement-norm water/cement = 0.5	2 x 15mm strand	50 tonne	holes drilled from sub-levels at H/W contact, first 2m debonded
13	Westmin H-W Mine	Cut & Fill	15m Open Stope	2m x 1.5m	51mm	portland cement-norm	2 x 15mm strand	50 tonne	
14	Eldorado Beaver Lodge	Cut & Fill	15m	1.8m x 2.4m	57mm	portland cement-norm bleeding additive	1 x 13mm strand		Spedel grout pump employed

Reference: Golder Associates

TABLE 5-5

MRMR ORIENTATION ADJUSTMENT FOR BACKS

Number of Joints Defining Block	Number of Faces Inclined away from Vertical and Adjustment Percentage				
	70%	75%	80%	85%	90%
3	3		2		
4	4	3		2	
5	5	4	3	2	1
6	6		4	3	2.1

MRMR ORIENTATION ADJUSTMENT FOR WALLS

Joint Condition Rating	Plunge of Joint Intersection and Adjustment Percentage		
0 - 5	10 - 30 = 85%	30 - 40 = 75	40 = 70%
5 - 10	10 - 20 = 90%	20 - 40 = 80	40 = 70%
10 - 15	20 - 30 = 85%	30 - 50 = 80	50 = 75%
15 - 20	30 - 40 = 90%	40 - 60 = 85	60 = 80/75%
20 - 30	30 - 50 = 90%	50 = 85%	
30 - 40	40 - 60 = 95%	60 = 90%	

MRMR ADJUSTMENT FOR BLASTING

Technique	Adjustment, %
Boring	100
Smooth Wall Blasting	97
Good Conventional Blasting	94
Poor Conventional Blasting	80

REFERENCE: LAUBSCHER, 1984.

TABLE 5-6: SUPPORT RECOMMENDATIONS FOR MRMR SYSTEM

	Support Techniques	
Rock reinforcement	a	bolts
	b	a + straps
	c	a + shotcrete
	d	mesh reinforced/steel fibre shotcrete + b as lateral restraint
	e	d + cable bolts as lateral restraint
	f	e + pinning
	g	spiling
	h	grouting
Rigid lining	i	timber sets
	j	rigid steel sets
	k	massive concrete
	l	j + concrete
	m	structurally reinforced concrete
Yielding lining	n	yielding steel arches
	o	yielding steel arches set in concrete/shotcrete
Backfill	p	backfill
Spalling control	q	bolts and rope laced mesh
Rock replacement	r	replace rock with material having greater compressive, shear and tensile strength

A number of additional comments by Laubscher are pertinent.

See Figure 9 for support selection charts.

Reference: Laubscher, 1984

TABLE 5-7: U.S. CORPS. OF ENGINEERS EMPIRICAL DESIGN

Parameter	Empirical Rule
<u>Minimum length and maximum spacing</u>	
Minimum length	Greatest of: (a) 2 x bolt spacing (b) 3 x thickness of critical and potentially unstable rock blocks (Note 1) (c) For elements above the springline: spans <6 m: 0.5 x span spans between 18 and 30 m: 0.25 x span spans between 6 and 18 m: interpolate between 3 and 4.5 m (d) For elements below the springline: height <18 m: as (c) above height >18 m: 0.2 x height
Maximum spacing	Least of: (a) 0.5 x bolt length (b) 1.5 x width of critical and potentially unstable rock bloc ks (Note 1) (c) 2.0 m (Note 2)
Minimum spacing	0.9 to 1.2 m
<u>Minimum average confining pressure</u>	
Minimum average confining pressure at yield point of elements (Note 3)	Greatest of: (a) Above springline: either pressure = vertical rock load of 0.2 x opening width or 40 kN/m^2 (b) Below springline: either pressure = vertical rock load of 0.1 x opening height or 40 kN/m^2 (c) At intersections: 2 x confining pressure determined above (Note 4)

Notes:

1. Where joint spacing is close and span relatively large, the superposition of two reinforcement patterns may be appropriate (e.g., long heavy elements on wide centres to support the span, and shorter, lighter bolts on closer centres to stabilize the surface against ravelling).

2. Greater spacing than 2.0 m makes attachment of surface support elements (e.g., weldmesh or chain link mesh) difficult.

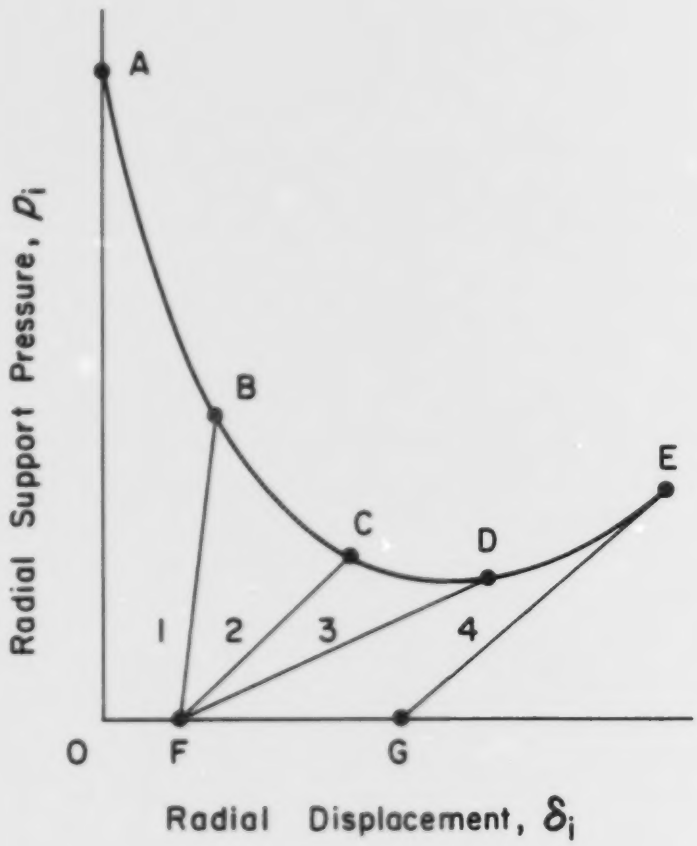
3. Assuming the elements behave in a ductile manner.

4. This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections and rock bolts are free to move toward both openings.

Reference: U.S. Corps. of Engineers (1980) and Douglas and Arthur (1983).

INFLUENCE OF SUPPORT STIFFNESS AND OF THE TIMING OF ITS INSTALLATION ON SUPPORT PERFORMANCE

Figure **5-1**



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SUPPORT STIFFNESS - PULLOUT TESTS

Figure 5-2

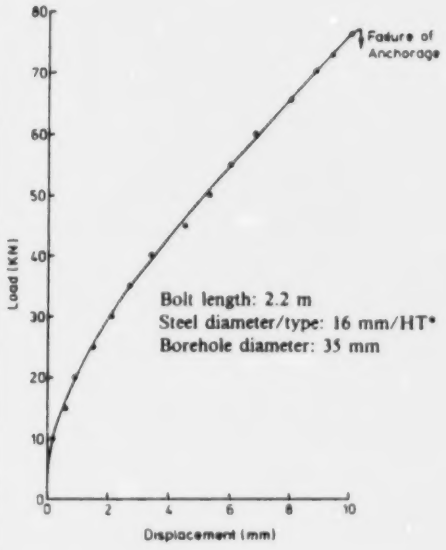

Fig. 10A Mechanically anchored rockbolt.

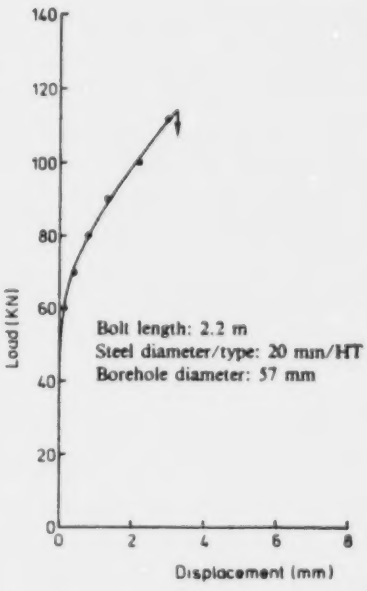

Fig. 10B Cement grouted rebar.

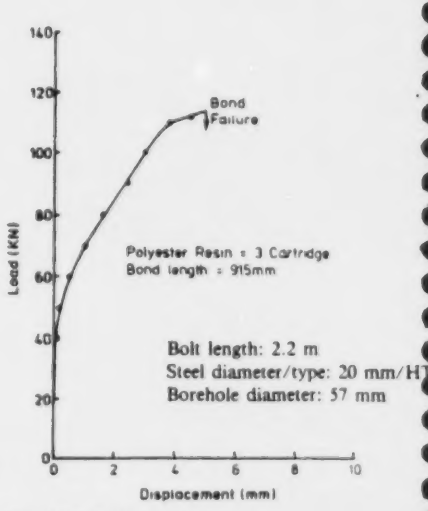

Fig. 10C Resin grouted rebar.

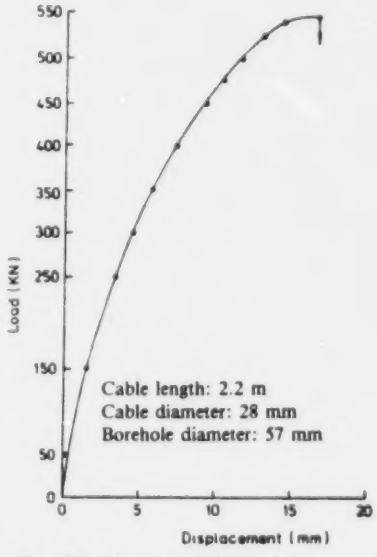

Fig. 10D Cement grouted Flexirope.

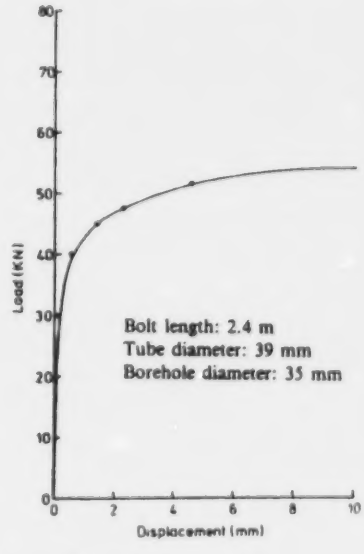

Fig. 10E Split Set.

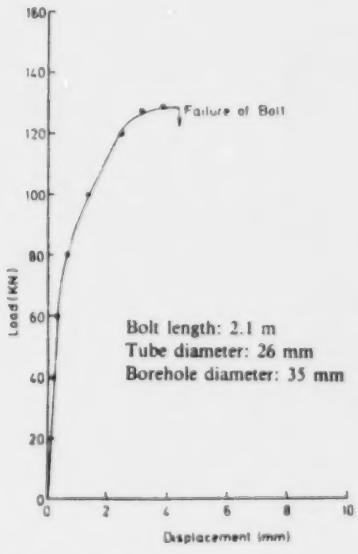

Fig. 10F Swellex.

* HT = High tensile steel

Note the different scales of the diagrams!

(after Stillborg, 1986)

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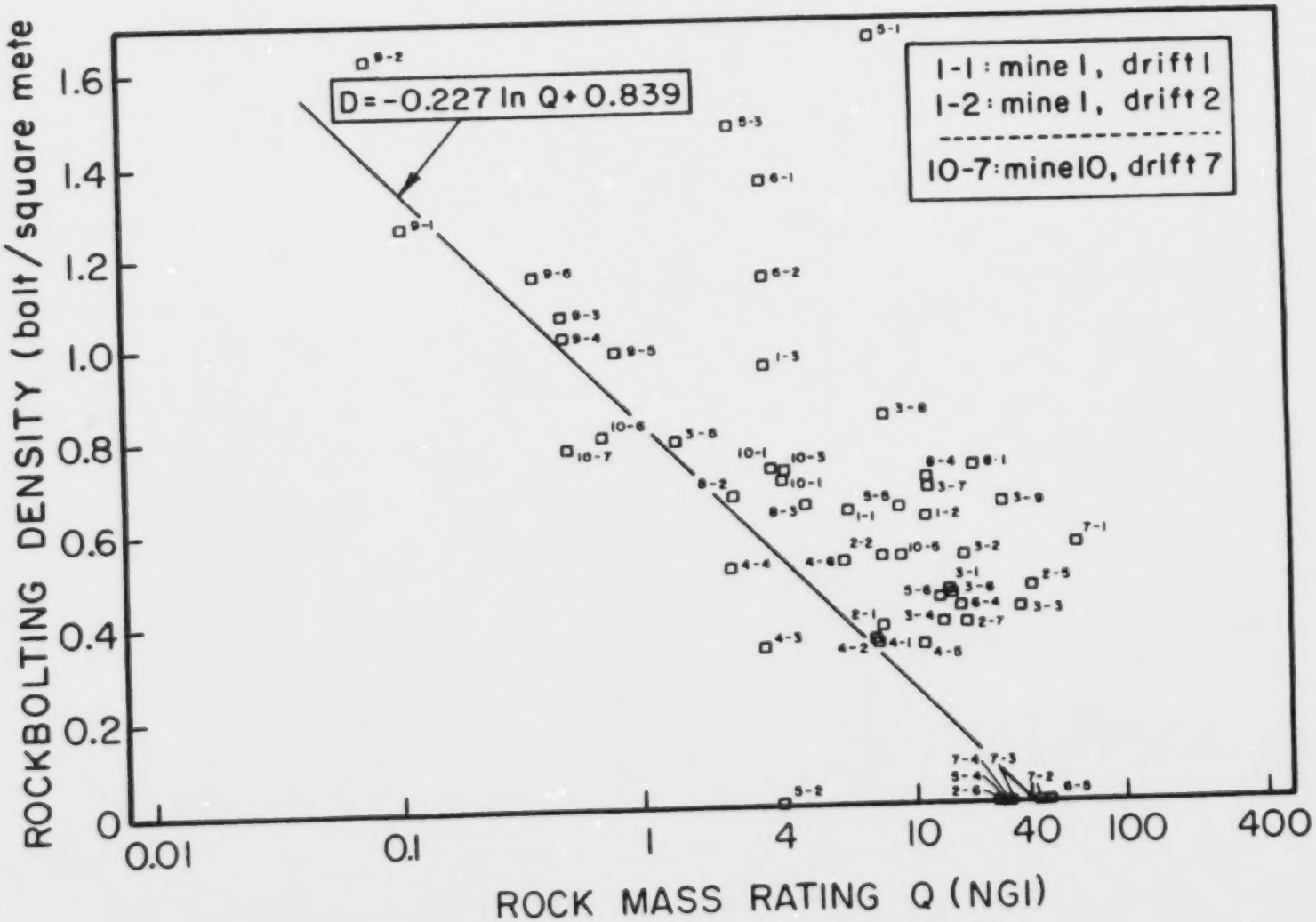


(after Choquet, 1988)

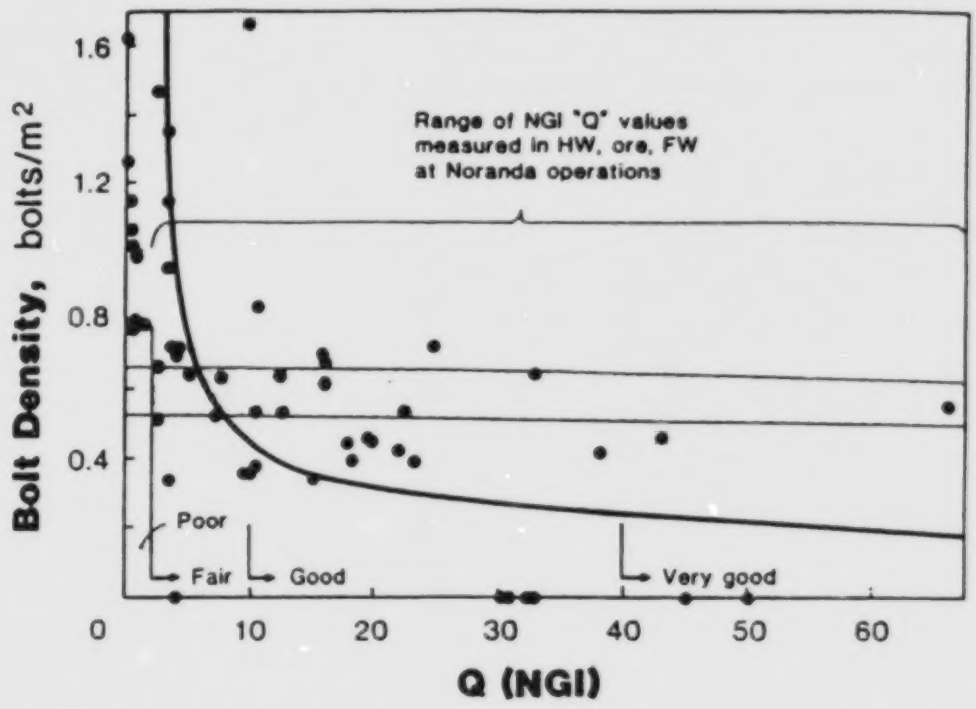
LINE OF MINIMUM BOLTING DENSITIES OBSERVED IN MINES OF THE CANADIAN SHIELD

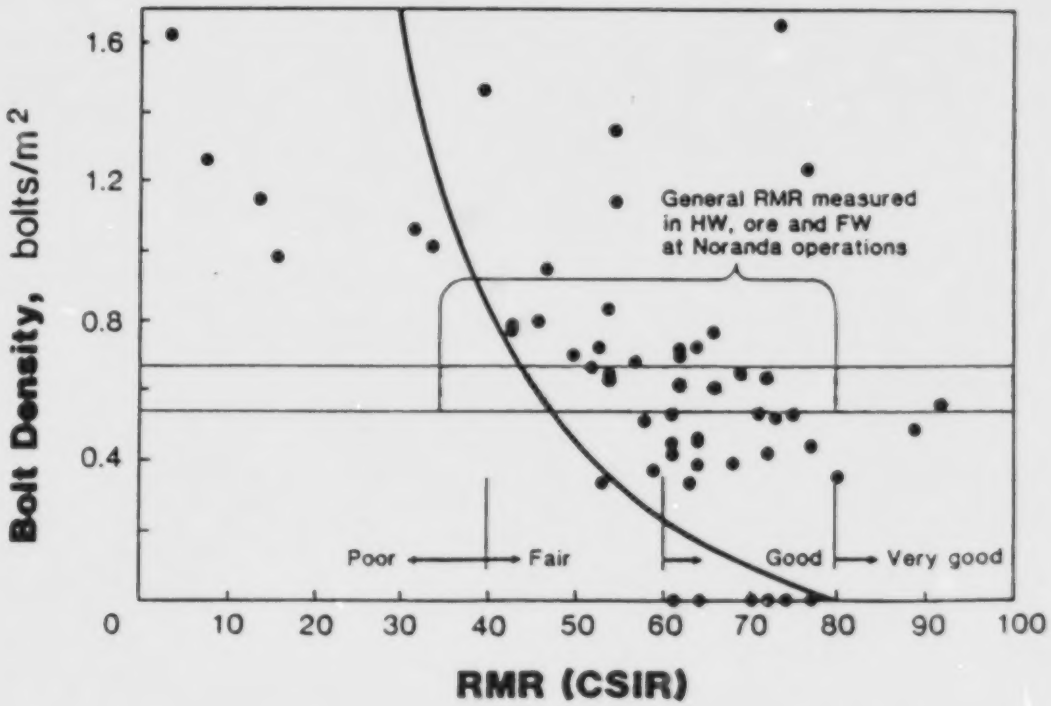
Figure **5-3**

Golder Associates

ROCKBOLT DENSITY VERSUS Q AND RMR

Figure **5-4**





(after Bawden et al, 1988)

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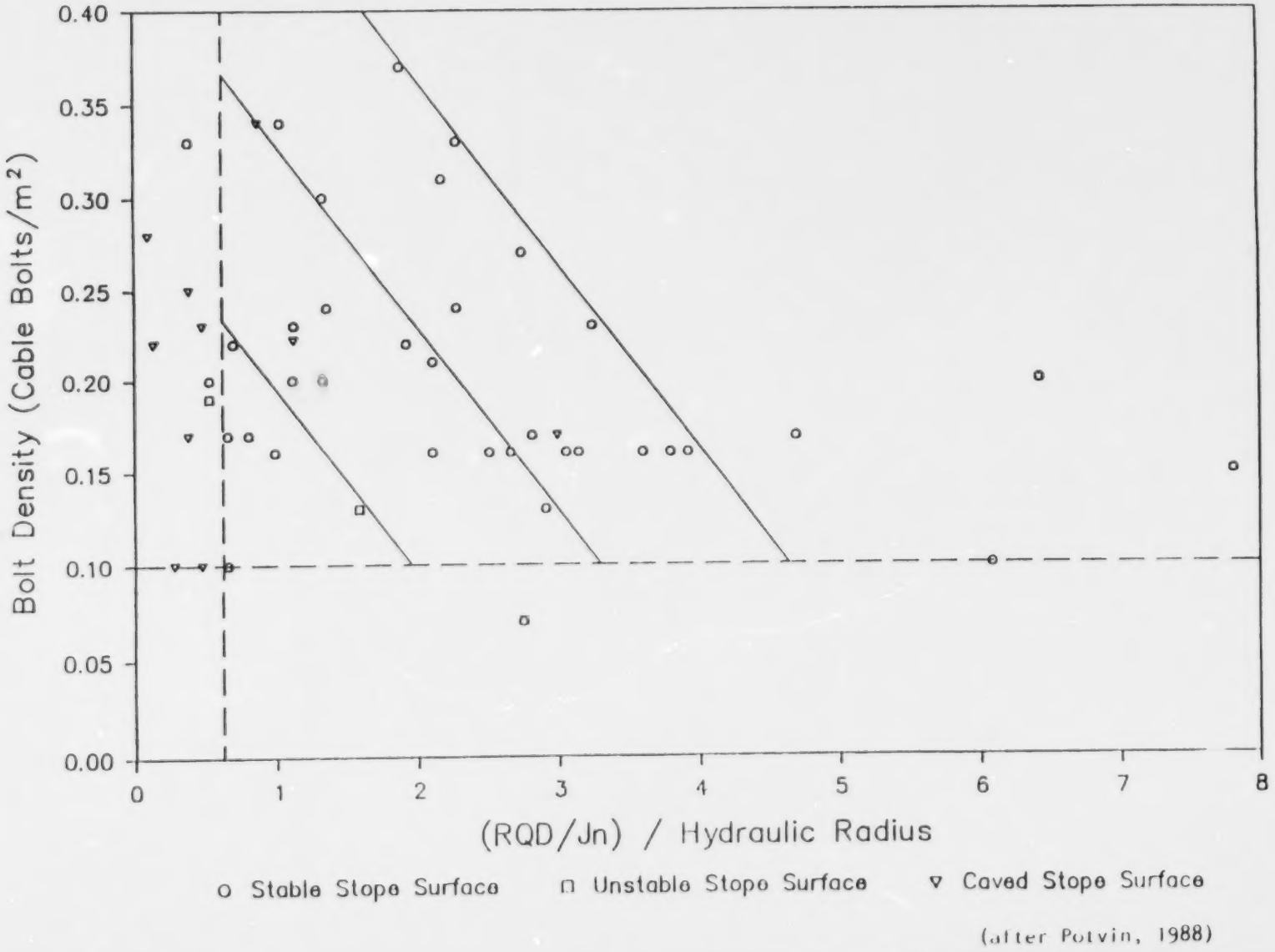

DESIGN CHART FOR CABLE BOLT DENSITY

Figure 5-5

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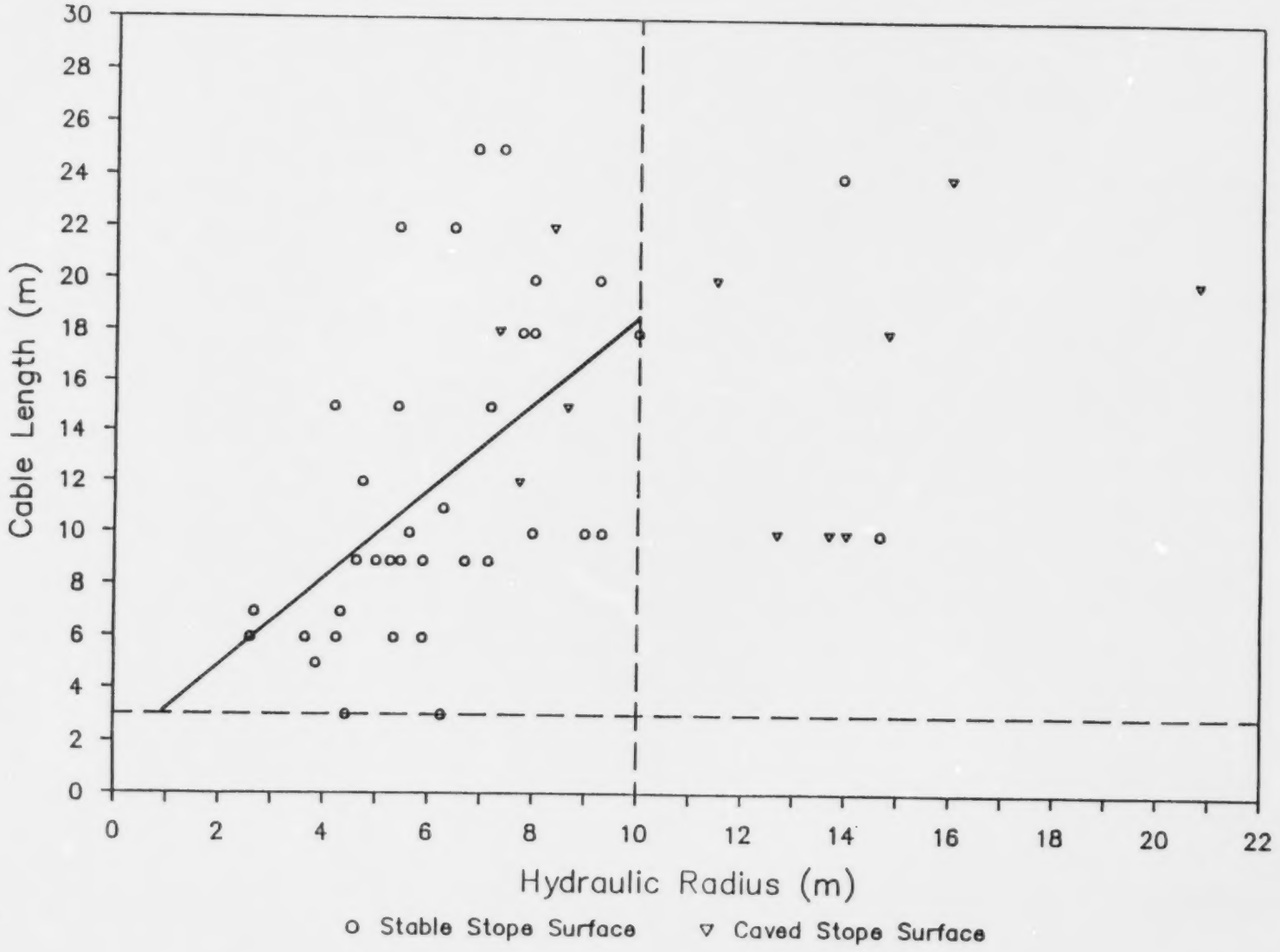


(after Potvin, 1988)

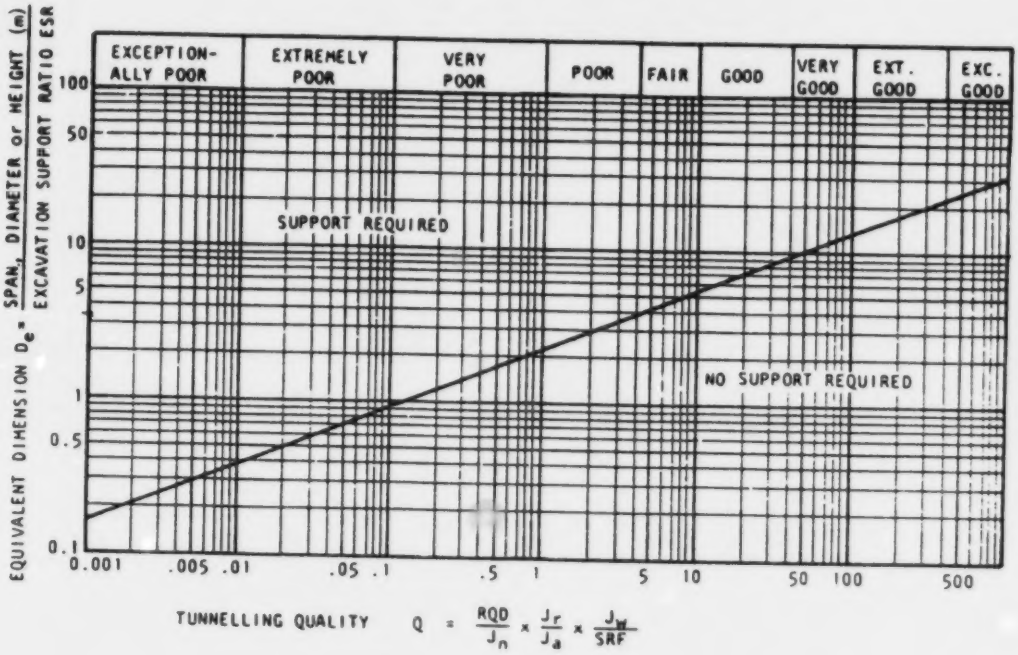
DESIGN CHART FOR CABLE BOLT LENGTH

Figure 5-6

Golder Associates

ROCK MASS RATING SUPPORT CLASSIFICATION

Figure **5–7**



Relationship between the maximum equivalent dimension D_e of an unsupported underground excavation and the NGI tunnelling quality index Q.

(after Barton et al, 1974)

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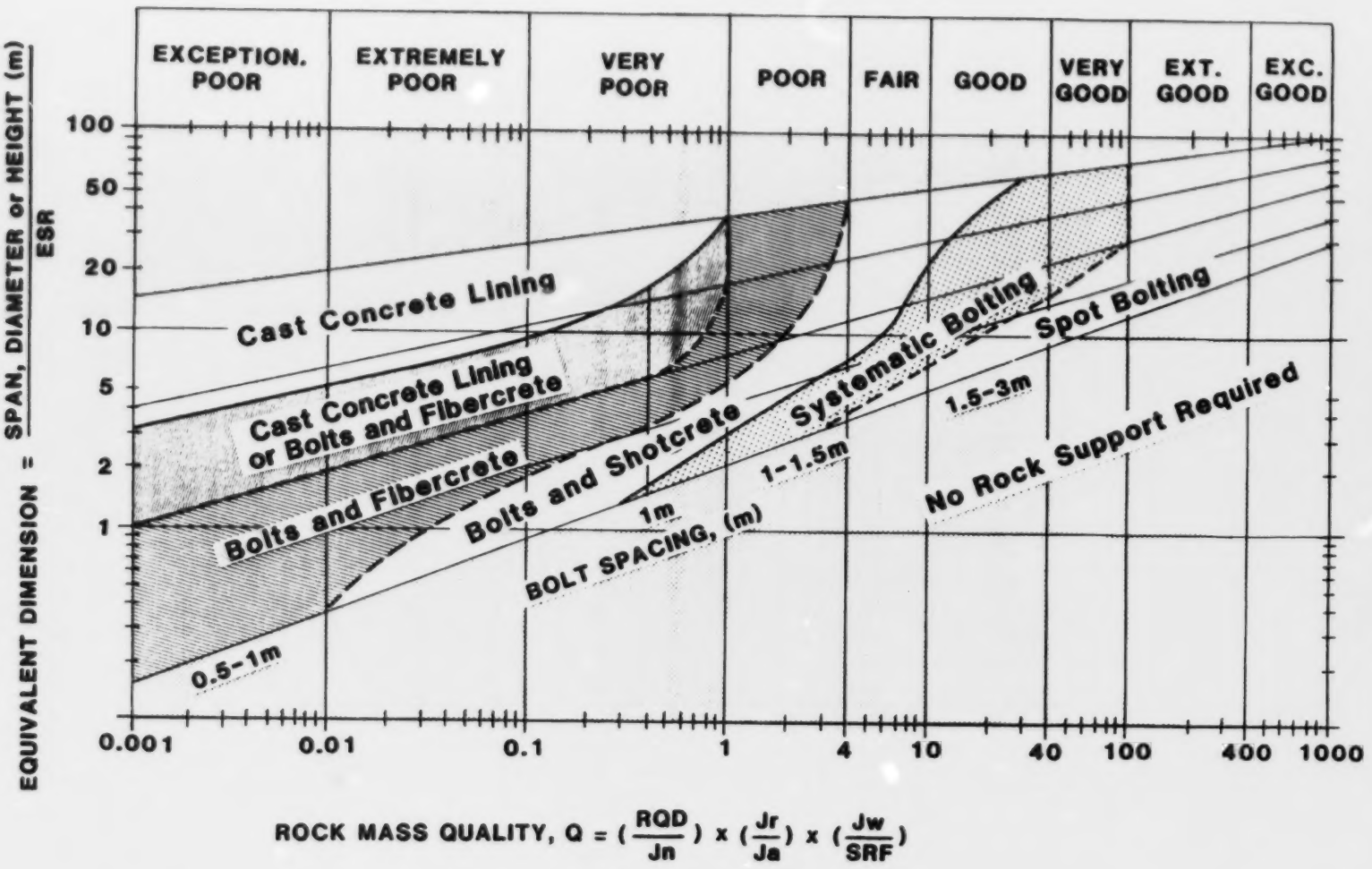

Reference : Barton, 1986

EMPIRICAL SUPPORT DESIGN USING Q

Figure 5-8

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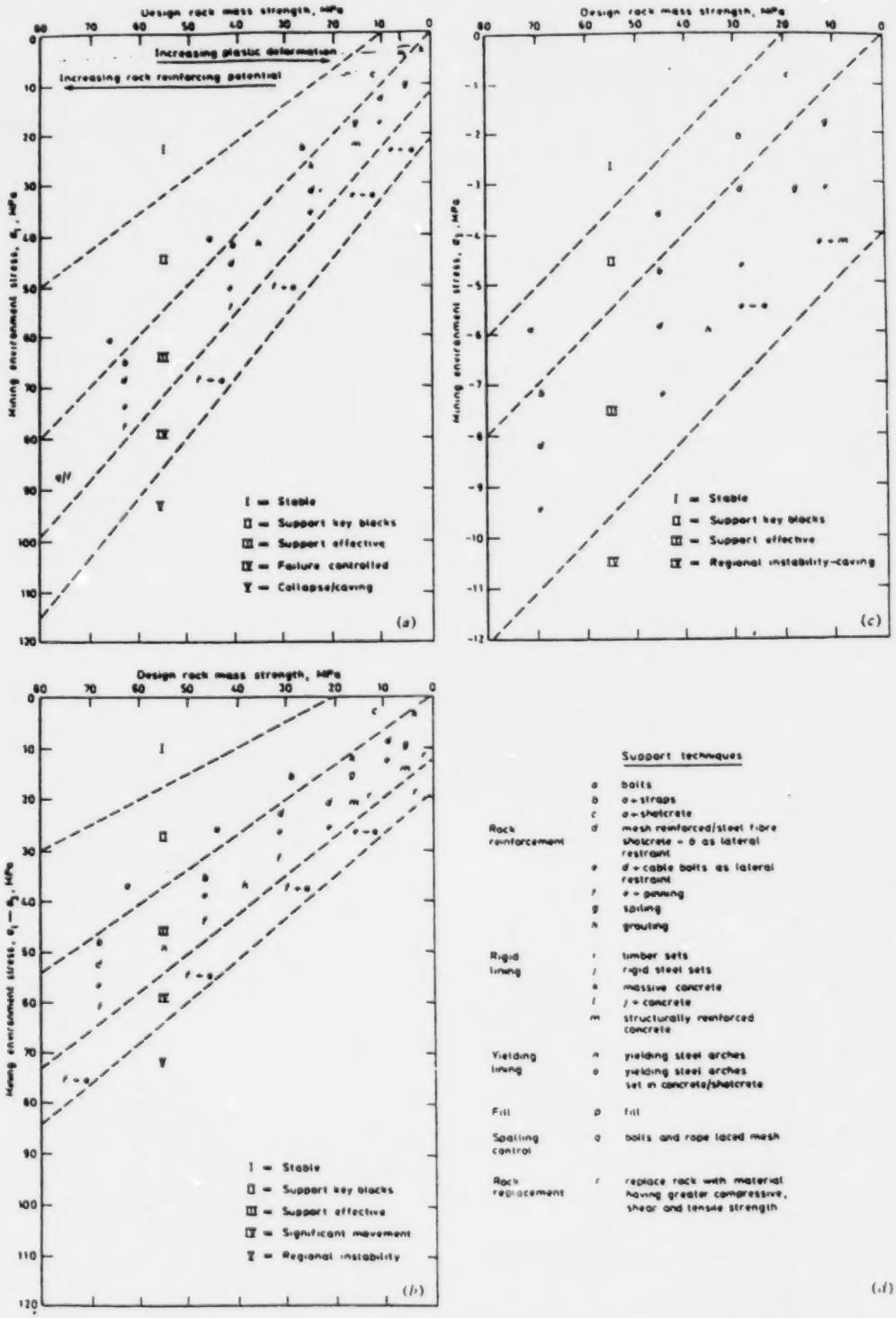
SUPPORT SELECTION CHARTS

Figure **5–9**



Support selection charts in which DRMS is related to (*a*) maximum stress, (*b*) difference between maximum and minimum compressive stresses and (*c*) minimum stress. Support techniques and symbols shown in (*d*).

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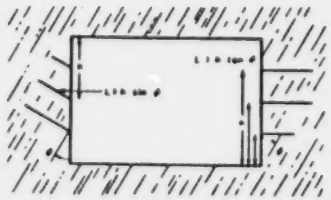
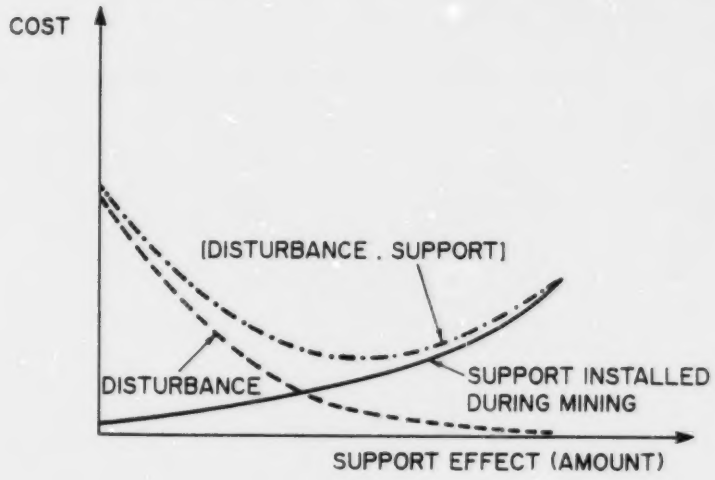
EXCAVATION SPAN (m)	NUMBER OF DISCONTINUITY SETS	BOLT DESIGN	COMMENTS
<15	≤2 inclined at 0-45° to horizontal	L = 0.3 B s = 0.5L (depending on thickness and strength of strata). Install bolts perpendicular to lamination where possible with wire mesh to prevent flaking	The purpose of bolting is to create a load-carrying beam over span. Fully bonded bolts create greater discontinuity shear stiffness. Tensioned bolts should be used in weak rock; subhorizontal tensioned bolts where vertical discontinuities occur
	≤2 inclined at 45-90° to horizontal	For side bolts: L > h sin ψ (if installed perpendicular to discontinuity); L > h tan ψ (if installed horizontally). See figure below for h and ψ; L = bolt length; s = bolt spacing; B = excavation span 	Roof bolting as above. Side bolts designed to prevent sliding along planar discontinuities. Spacing should be such that anchorage capacity is greater than sliding or toppling weight. Bolts should be tensioned sufficiently to prevent sliding
	≥3 with clean tight interfaces	L = 2s s = 3-4 x block dimension. Install bolts perpendicular to excavation periphery with wire mesh to prevent flaking	Bolts should be installed quickly after excavation to prevent loosening and retain tangential stresses. Prestresses should be applied to create zone of radial confinement. Sidewall bolting where toe of wedge daylights in side wall
15	≤2	L_1 = 0.3 B_1 primary bolting s_1 = 0.5 L_1 L_2 = 0.3 s_1 secondary bolting s_2 = 0.5 L_2 install wire mesh to prevent spalling	Primary bolting conforms to smaller excavation design. Secondary (and tertiary) bolting supplements primary design (See figure below)
	≥3 with clean tight interfaces	L_1 = 0.3 B_1 primary bolting s_1 = 0.5 L_1 s_2 = 3-4 x block size; secondary bolting L_2 = 2 s_2	Primary bolting should have sufficient capacity to restrain major blocks. Decisions on block size for secondary bolting should be left to the section engineer

Figure 5-10

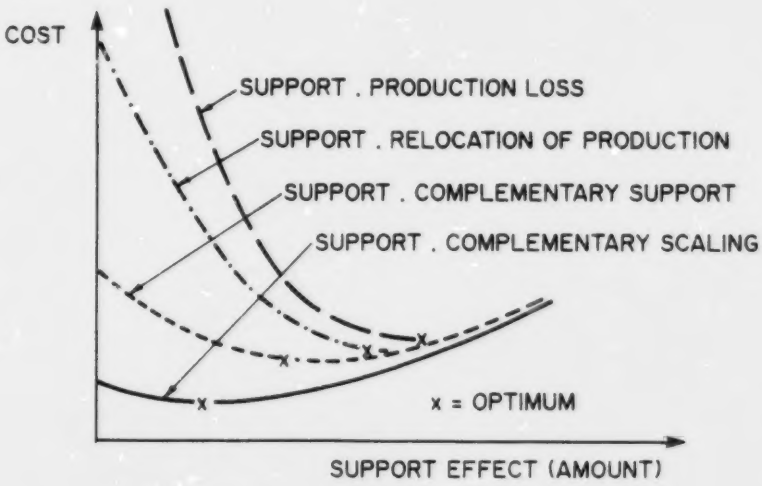
ECONOMIC OPTIMIZATION OF GROUND SUPPORT

Figure **5-11**

COST OF SUPPORT INSTALLATION AND POTENTIAL DISTURBANCE VS. AMOUNT OF GROUND SUPPORT



ECONOMIC RISK OF OPERATIONAL DISTURBANCES VS. AMOUNT OF GROUND SUPPORT



Reference : Krauland, 1984

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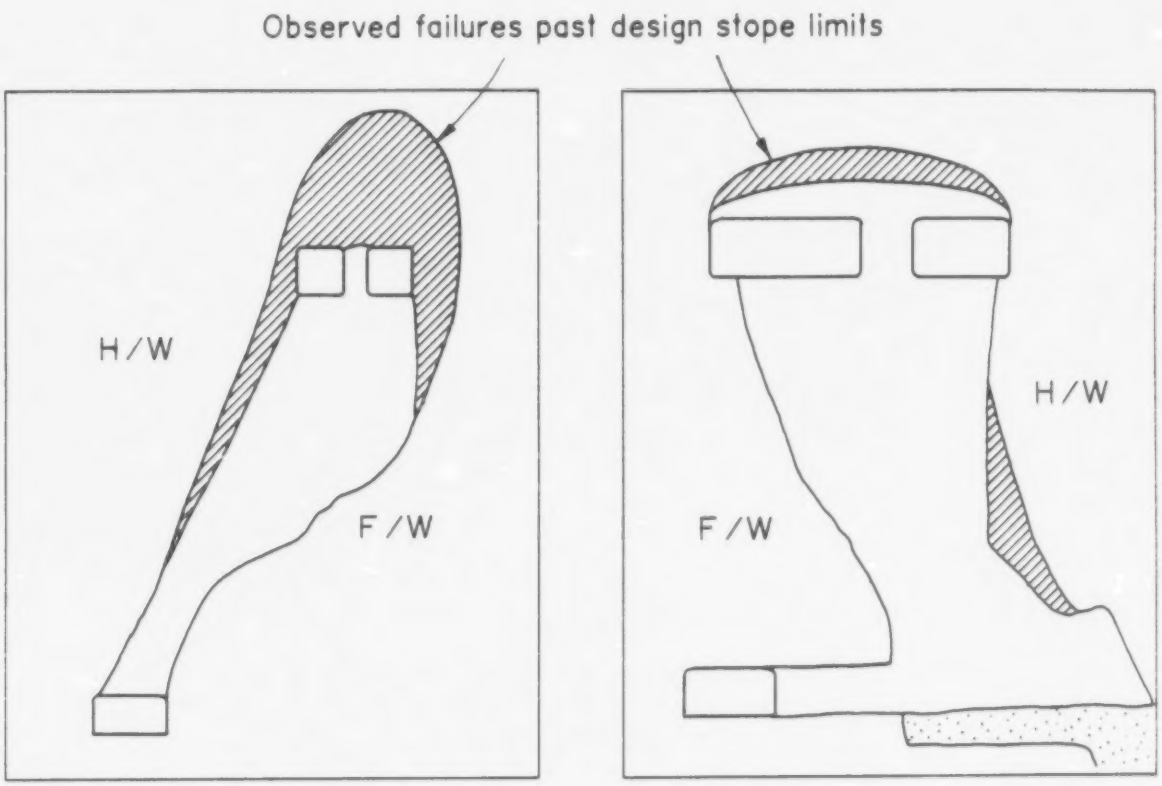
TYPICAL EXAMPLES OF OBSERVED BACK AND HANGINGWALL FAILURES THOMPSON MINE

Figure **5-12**



Not to Scale

Reference : Inco Ltd.

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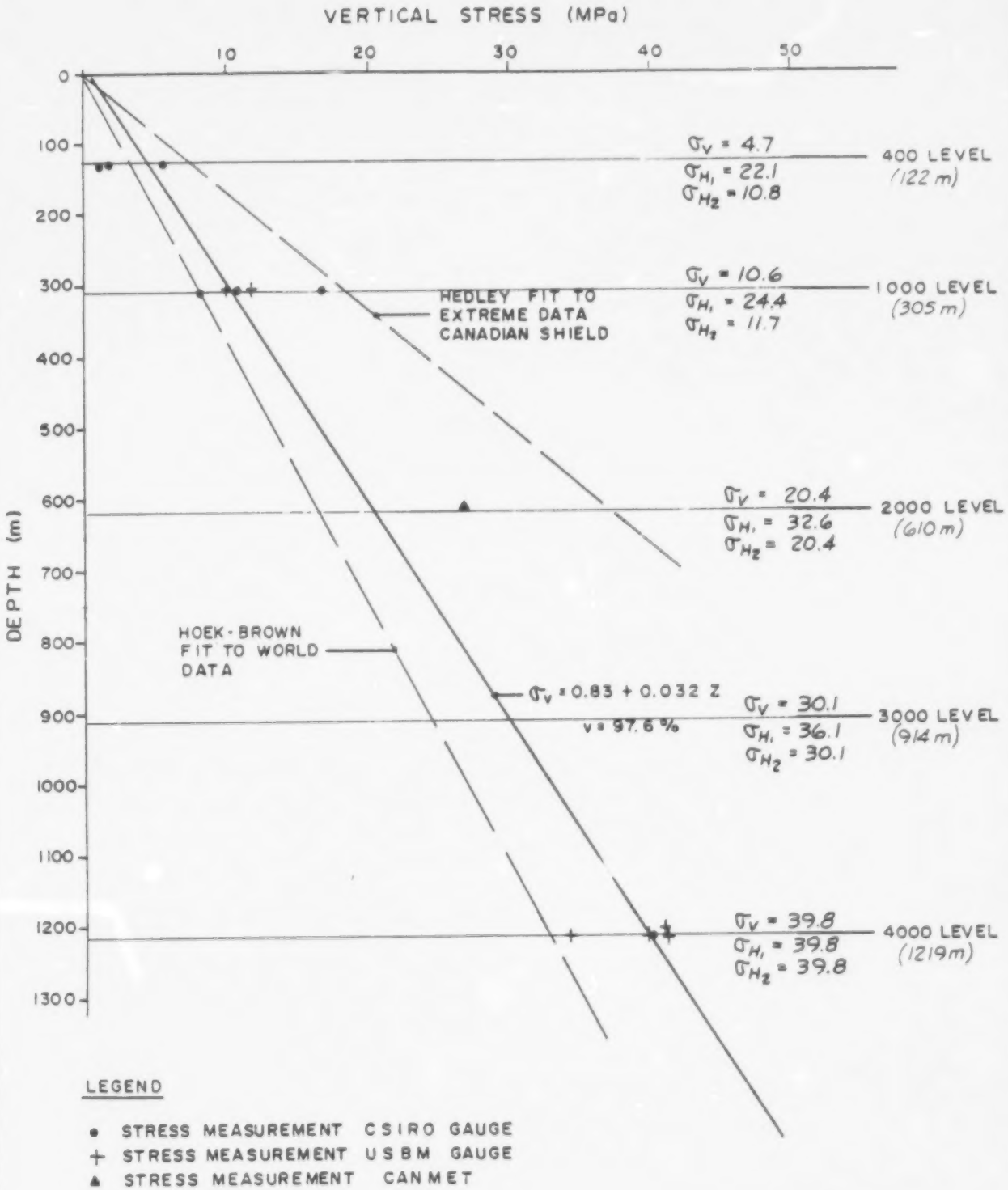
ESTIMATED VERTICAL STRESS GRADIENT THOMPSON MINE

Figure **5-13**



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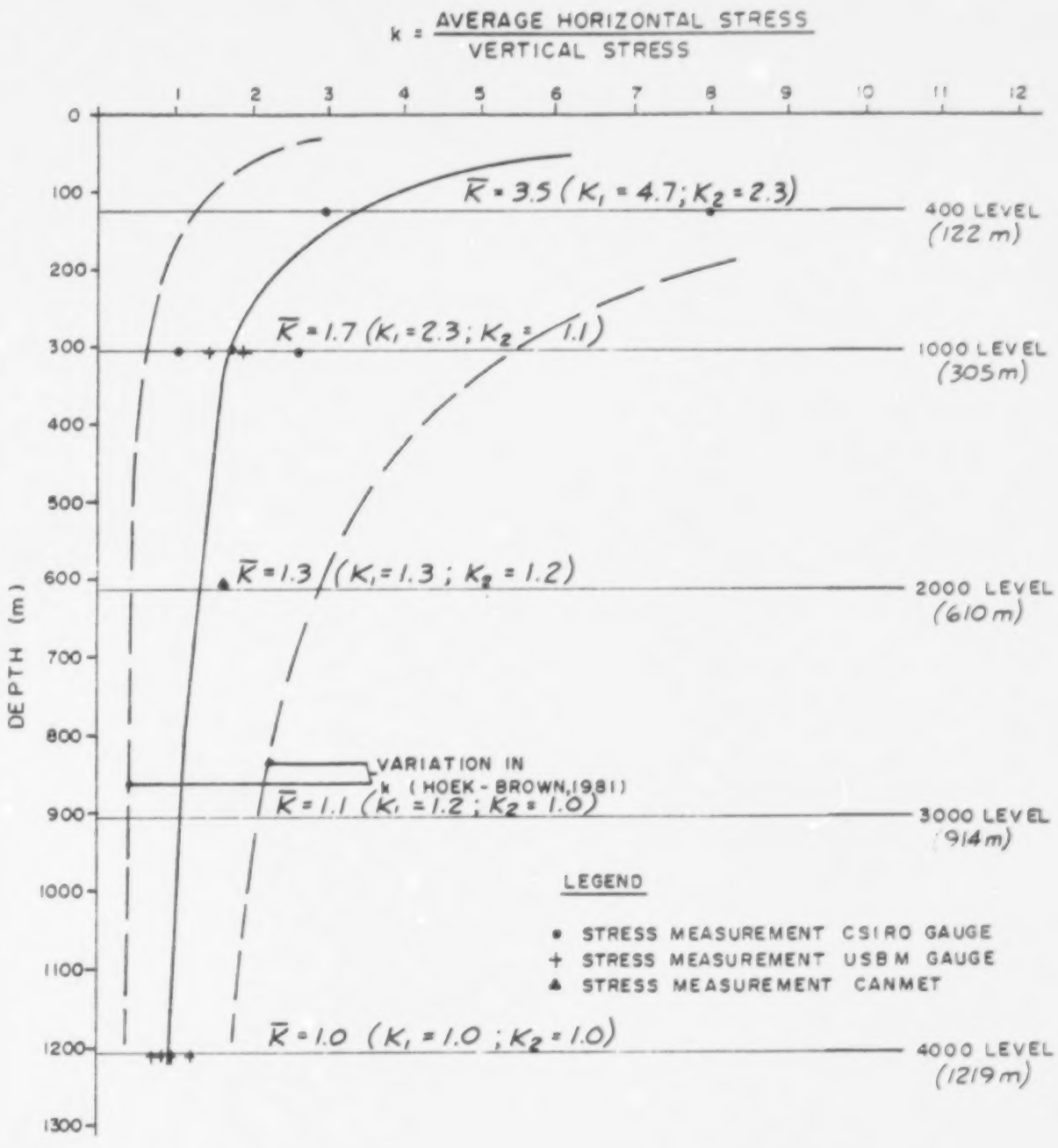
Golder Associates

ESTIMATED K-RATIO GRADIENT
THOMPSON MINE

Figure **5–14**


k = AVERAGE HORIZONTAL STRESS / VERTICAL STRESS
1 2 3 4 5 6 7 8 9 10 11 12
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300
DEPTH (m)
$\bar{K}$ = 3.5 (K_1 = 4.7; K_2 = 2.3)
400 LEVEL (122 m)
$\bar{K}$ = 1.7 (K_1 = 2.3; K_2 = 1.1)
1000 LEVEL (305m)
$\bar{K}$ = 1.3 (K_1 = 1.3 ; K_2 = 1.2)
2000 LEVEL (610 m)
VARIATION IN k (HOEK - BROWN, 1981)
$\bar{K}$ = 1.1 (K_1 = 1.2 ; K_2 = 1.0)
3000 LEVEL (914 m)
LEGEND
• STRESS MEASUREMENT CSIRO GAUGE
+ STRESS MEASUREMENT USBM GAUGE
▲ STRESS MEASUREMENT CANMET
$\bar{K}$ = 1.0 (K_1 = 1.0 ; K_2 = 1.0)
4000 LEVEL (1219 m)

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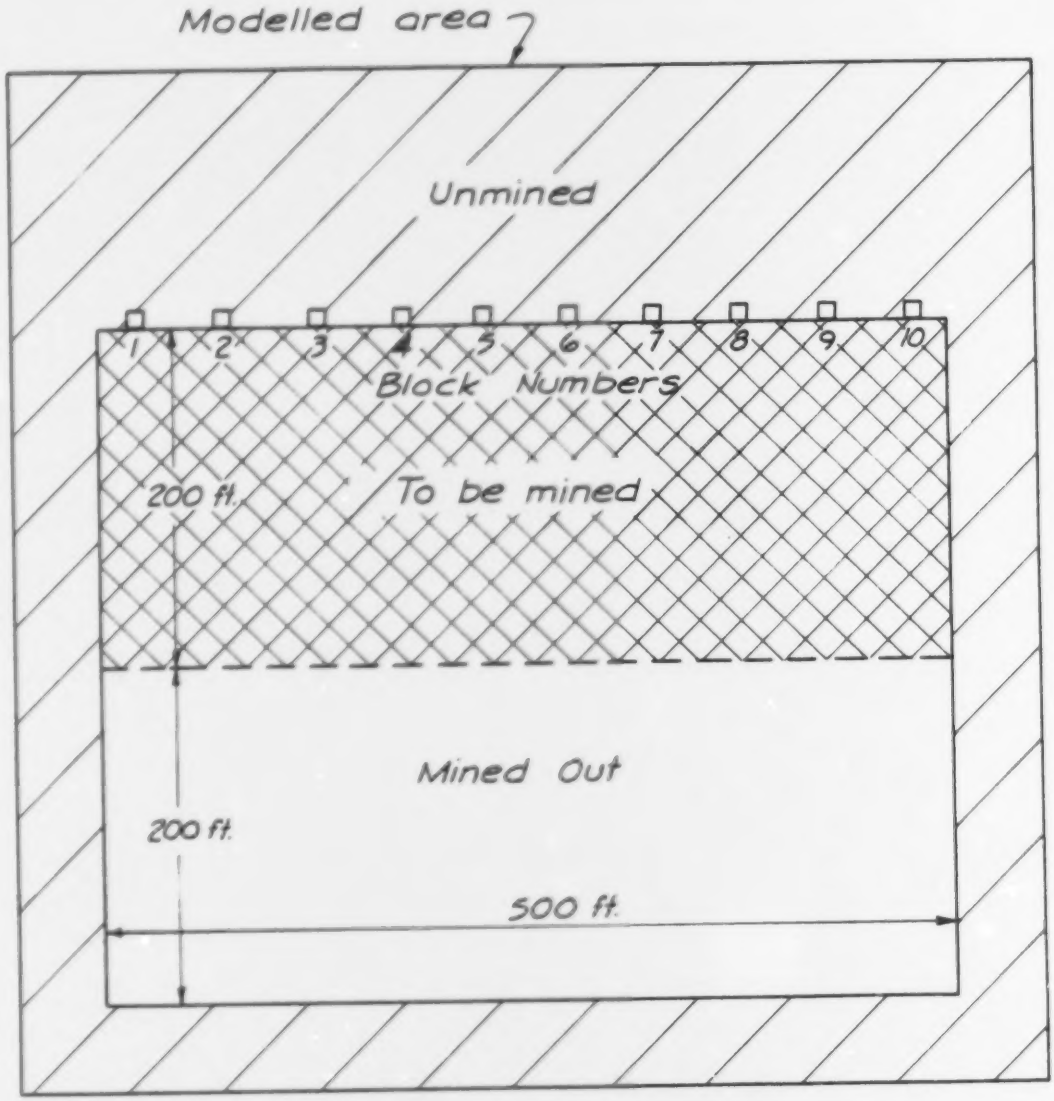
VIRGIN MINING - NFOLD MODEL | Figure **5-15**

LONGITUDINAL SECTION



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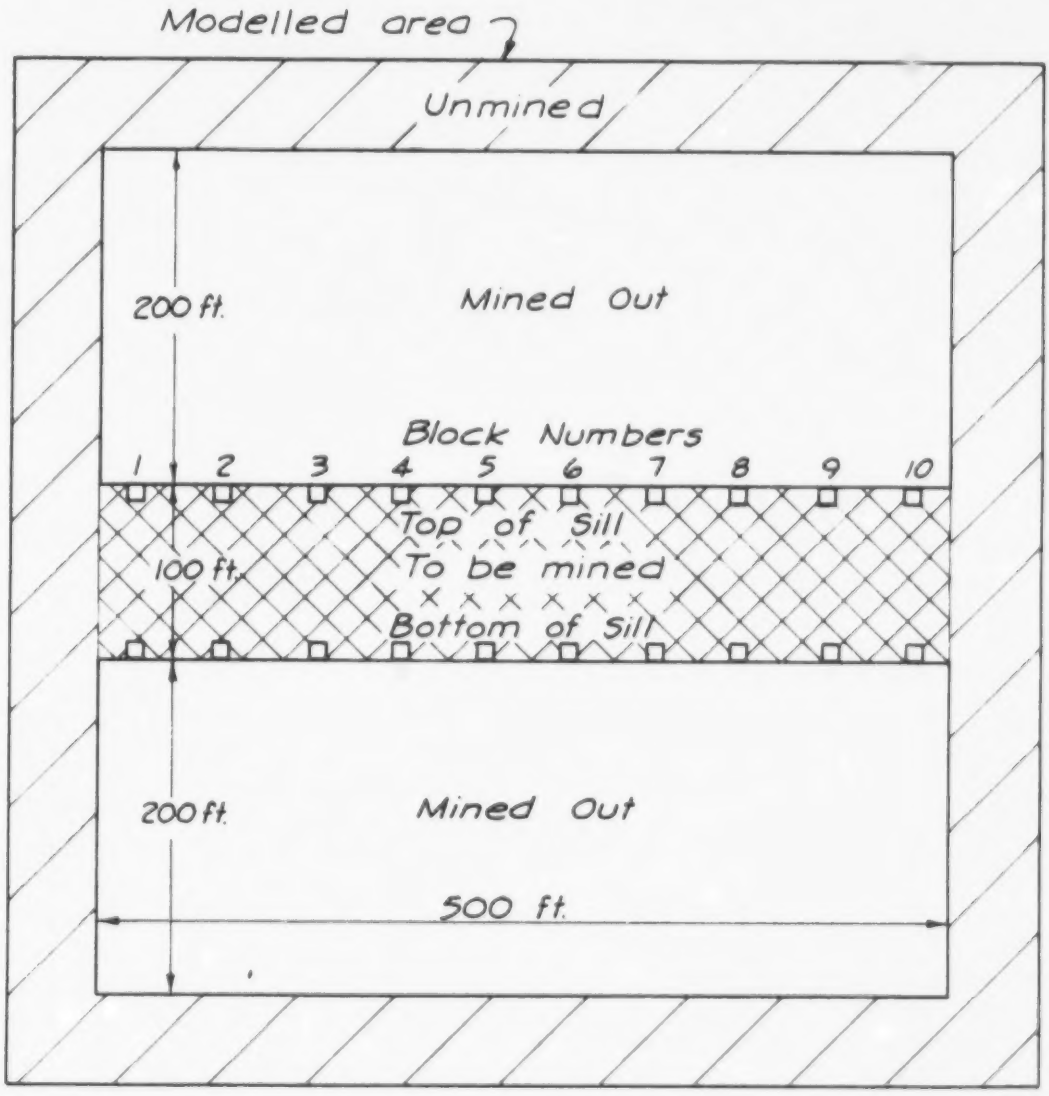
REMNANT MINING - NFOLD MODEL | Figure **5-16**

LONGITUDINAL SECTION



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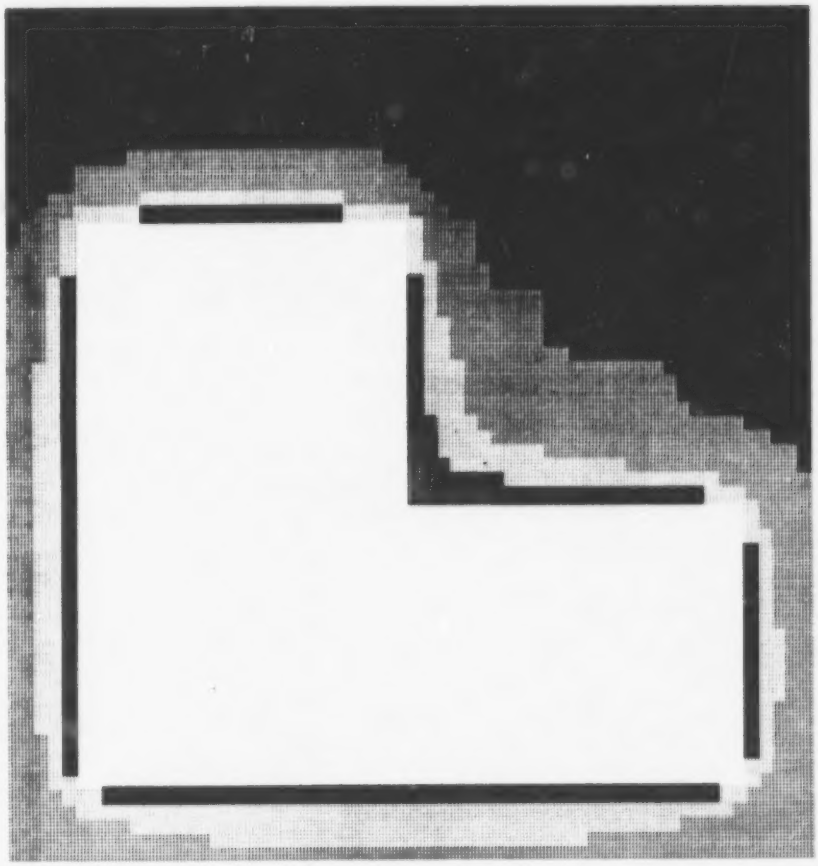
VIRGIN MINING - FIFTH SLICE
2000 FT. DEPTH

Figure **5-17**

LONGITUDINAL SECTION



Element size = 10 ft. x 10 ft.

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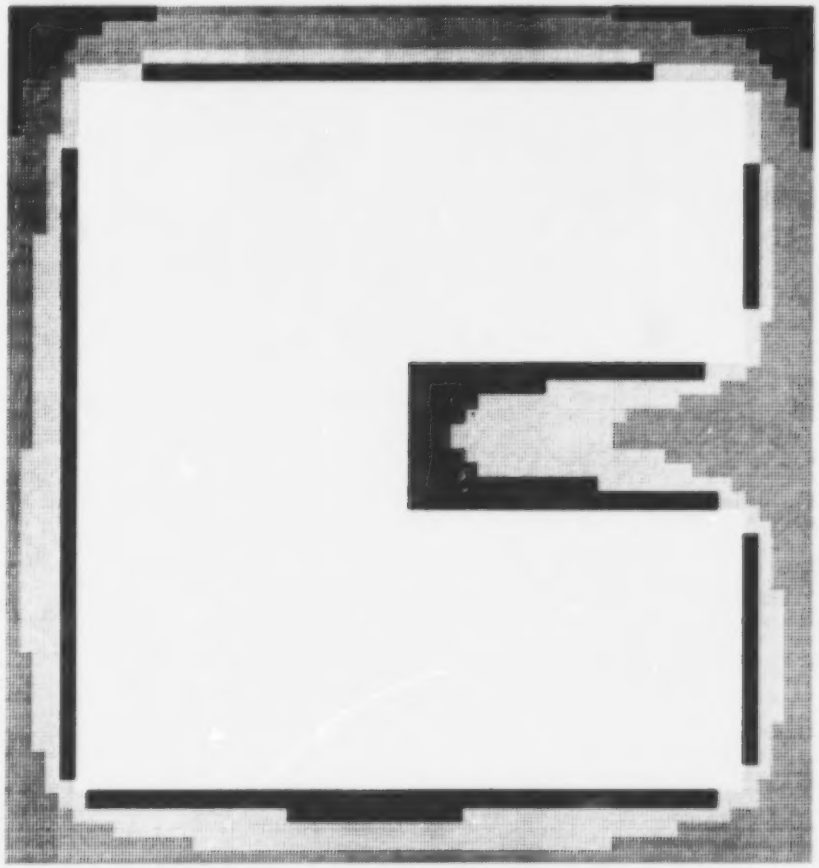
REMNANT MINING - FIFTH SLICE
2000 FT DEPTH

Figure **5-18**

LONGITUDINAL SECTION



Legend
Normal Stress
(MPa)

- 10 < 25
- 25 < 35
- 35 < 50
- 50 < 70
- 70 < 100
- 100 >

Element size = 10 ft. x 10 ft.

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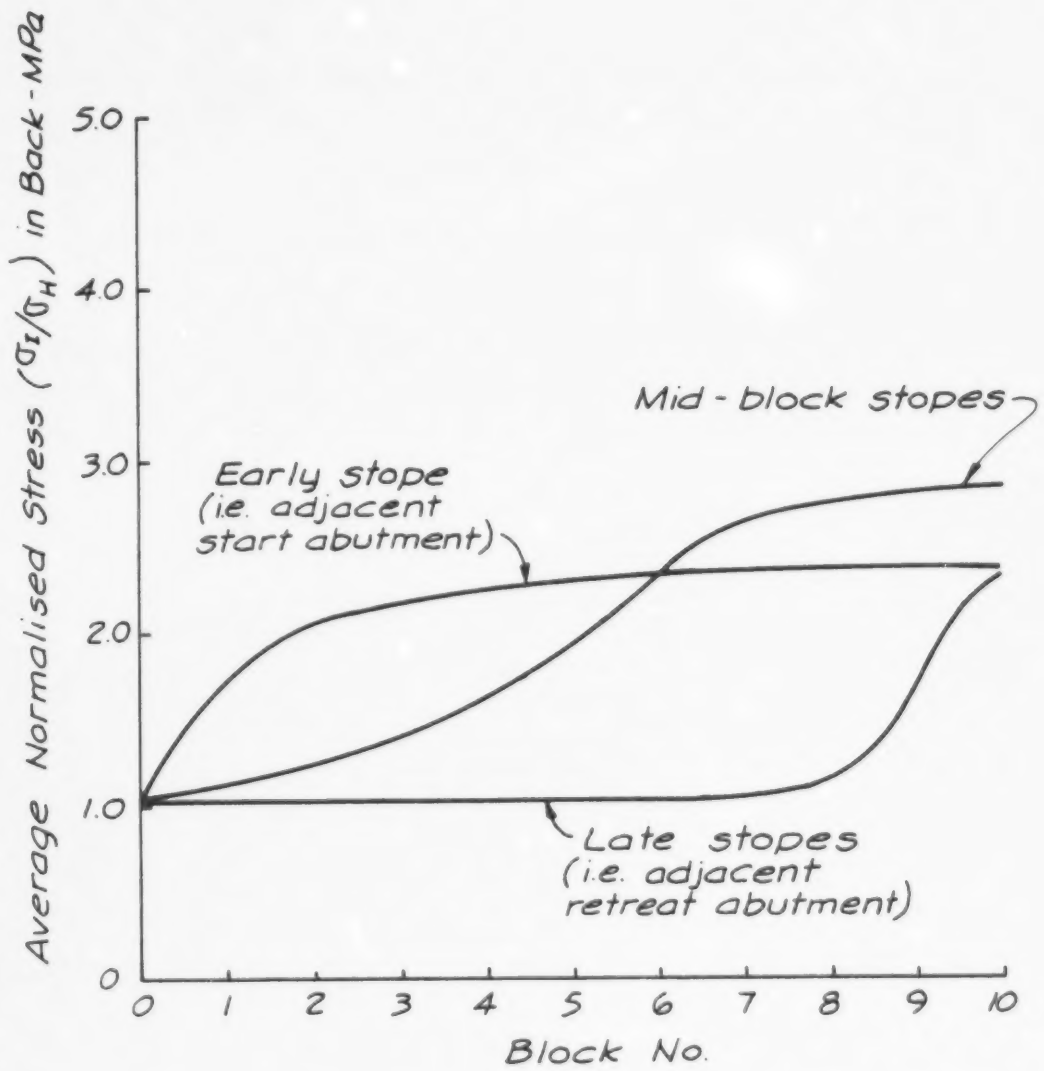
GENERALIZED STRESS HISTORY
STOPE BACK
VIRGIN MINING

Figure **5-19**



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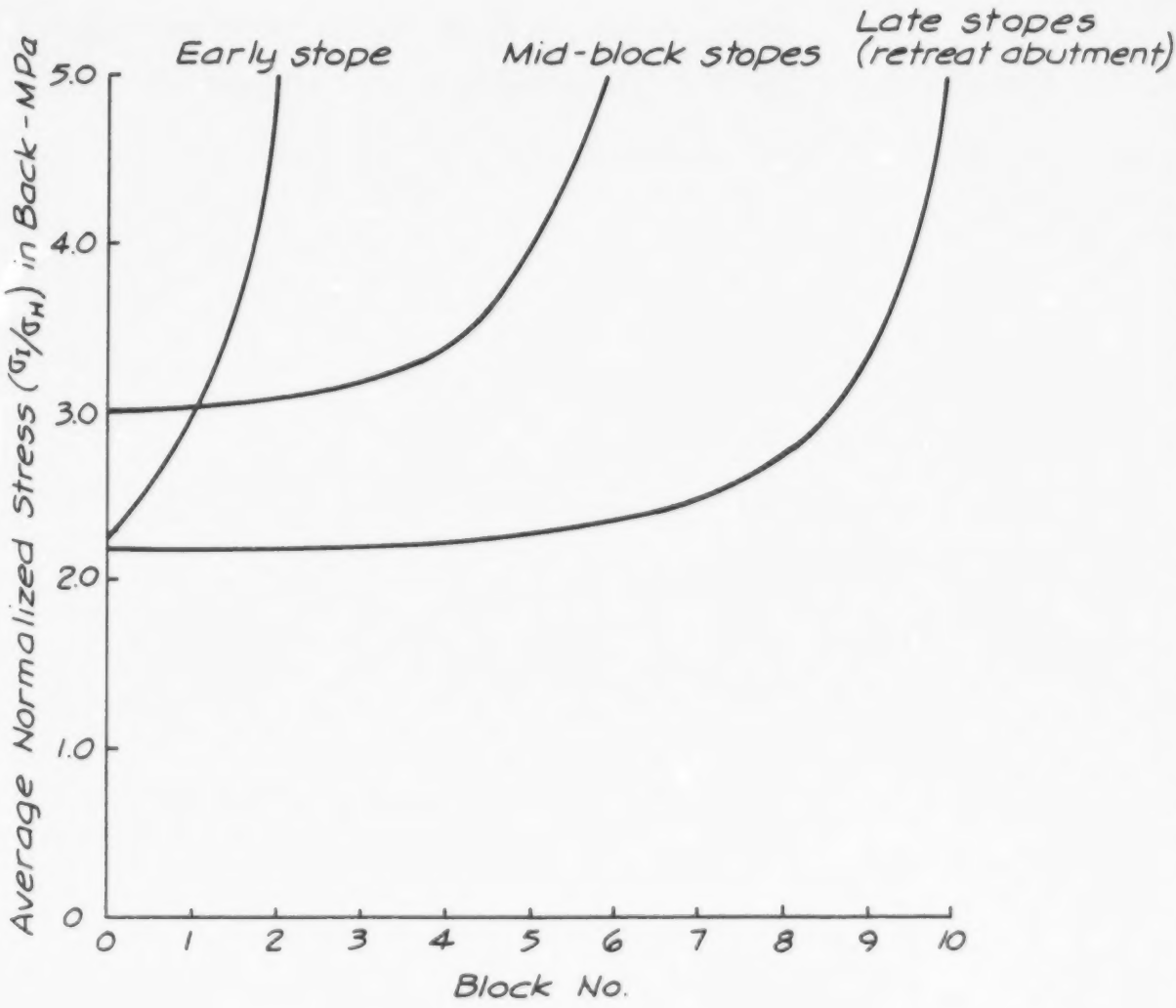
GENERALIZED STRESS HISTORY
TOP AND BOTTOM OF SILL
REMNANT MINING

Figure **5–20**



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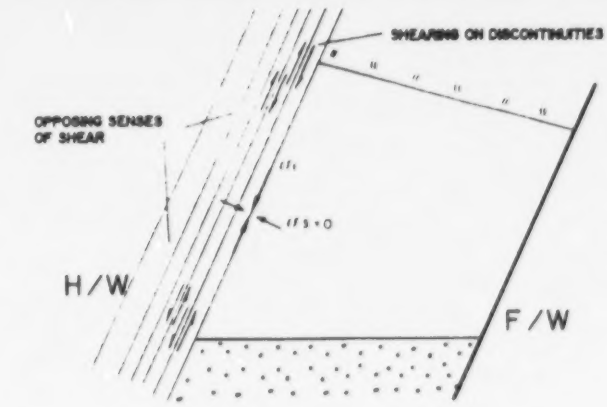
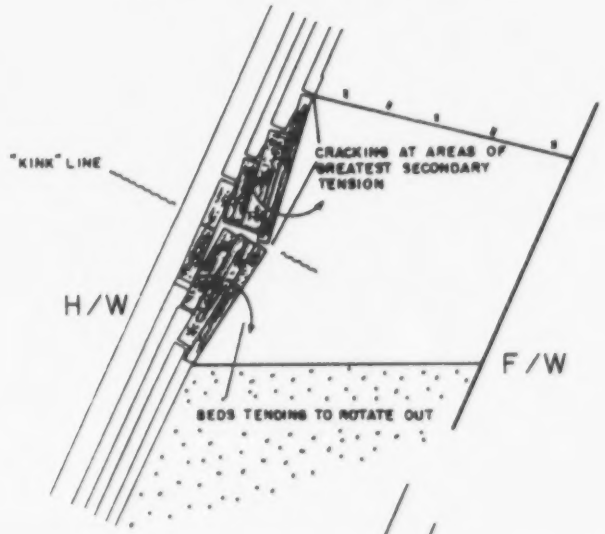
EXAMPLE OF WALL FAILURE DUE TO SHEARING ALONG DISCONTINUITIES

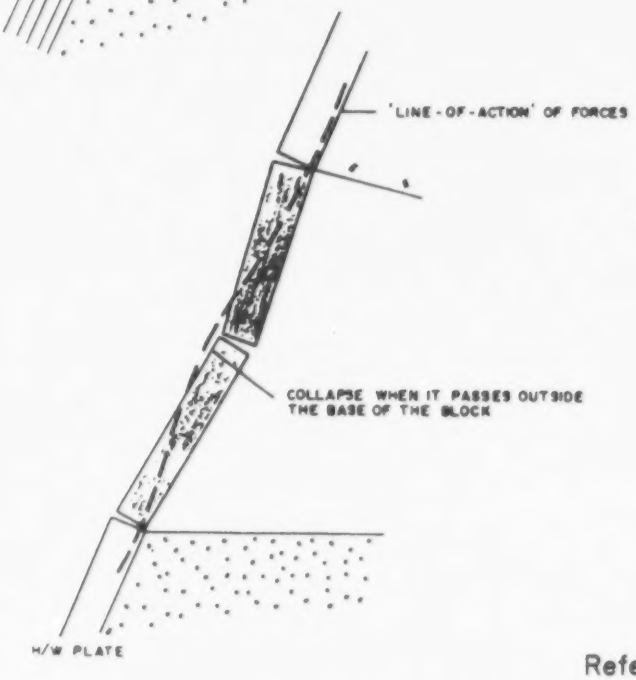
Figure **5-21**



(a) SHEARING



(b) 'KINK' DEVELOPMENT



(c) CONDITION FOR COLLAPSE

Reference: Lee, 1981

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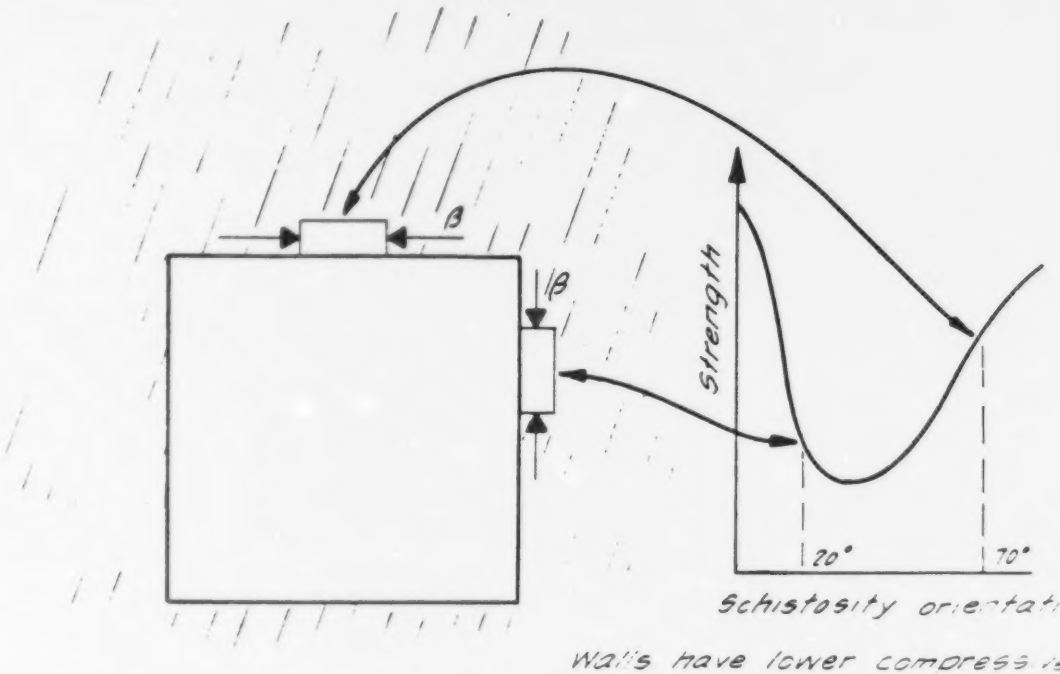
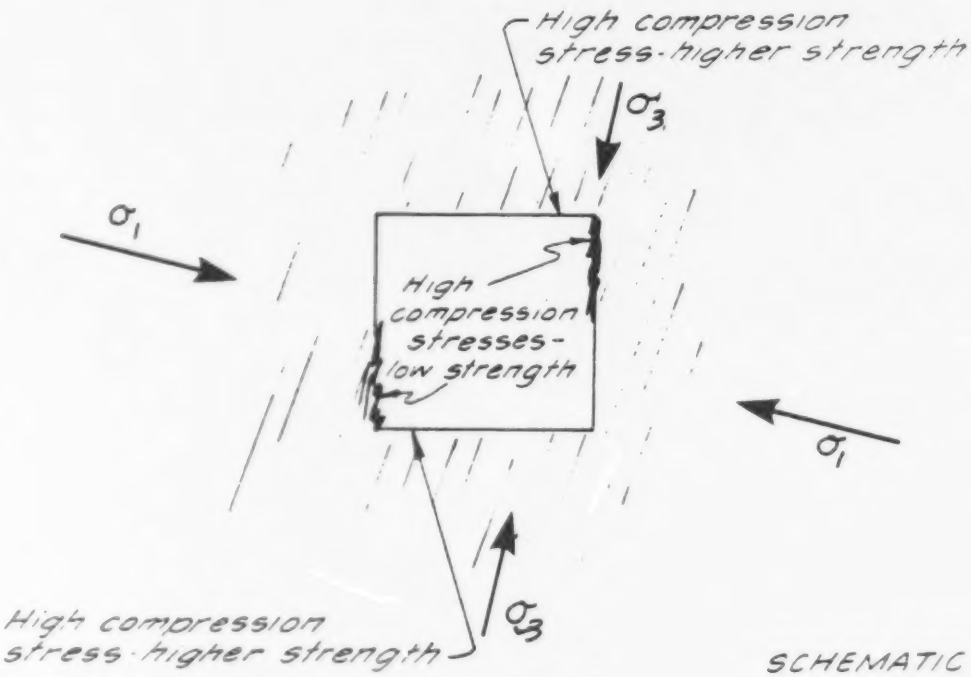
PREFERENTIAL FRACTURING OF WALLS AS A RESULT OF SCHISTISITY

Figure **5-22**



Walls have lower compressive strength than back due to orientation of schistosity relative to loading direction



SCHEMATIC ONLY
Not to Scale

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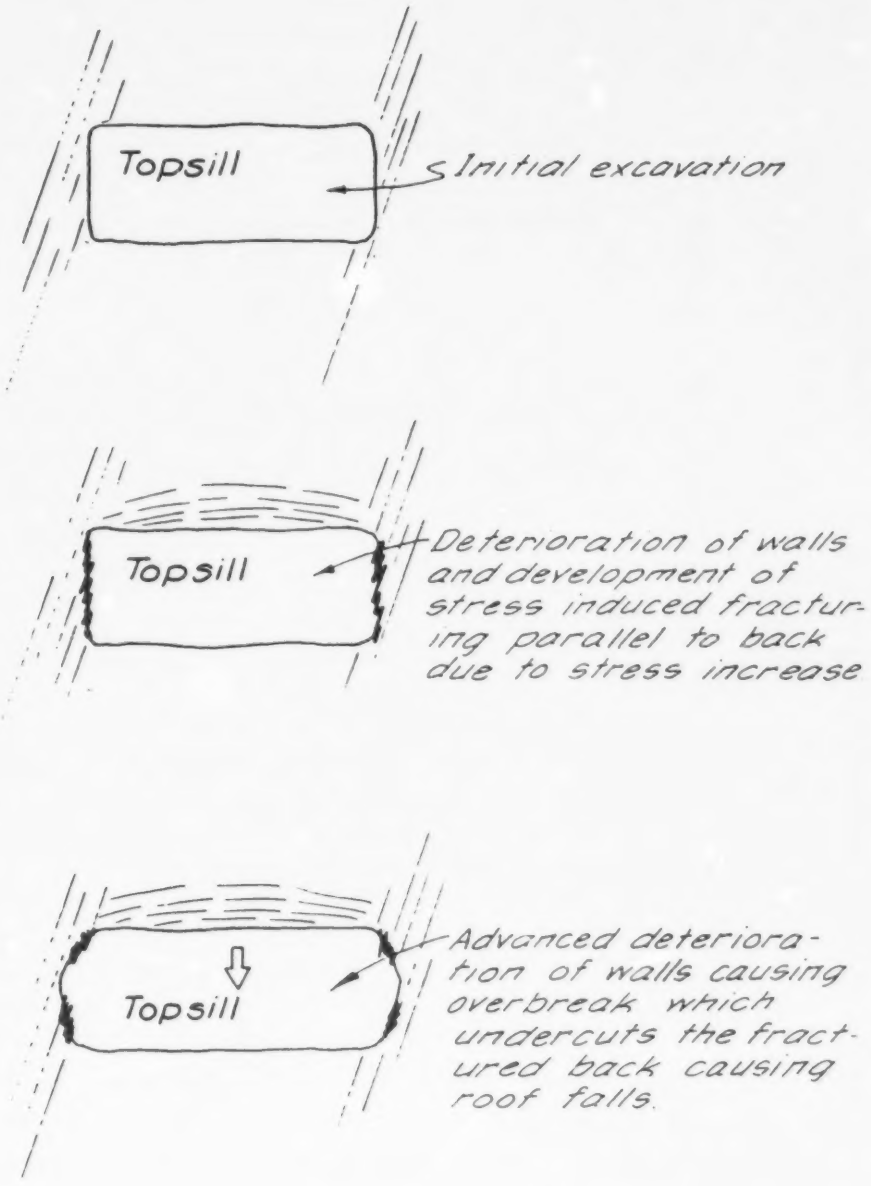
UNDERCUTTING OF BACK DUE TO PROGRESSIVE DETERIORATION OF WALLS

Figure **5-23**


Topsill
Initial excavation
Topsill
Deterioration of walls and development of stress induced fracturing parallel to back due to stress increase
Topsill
Advanced deterioration of walls causing overbreak which undercuts the fractured back causing roof falls.

SCHEMATIC ONLY
Not to scale

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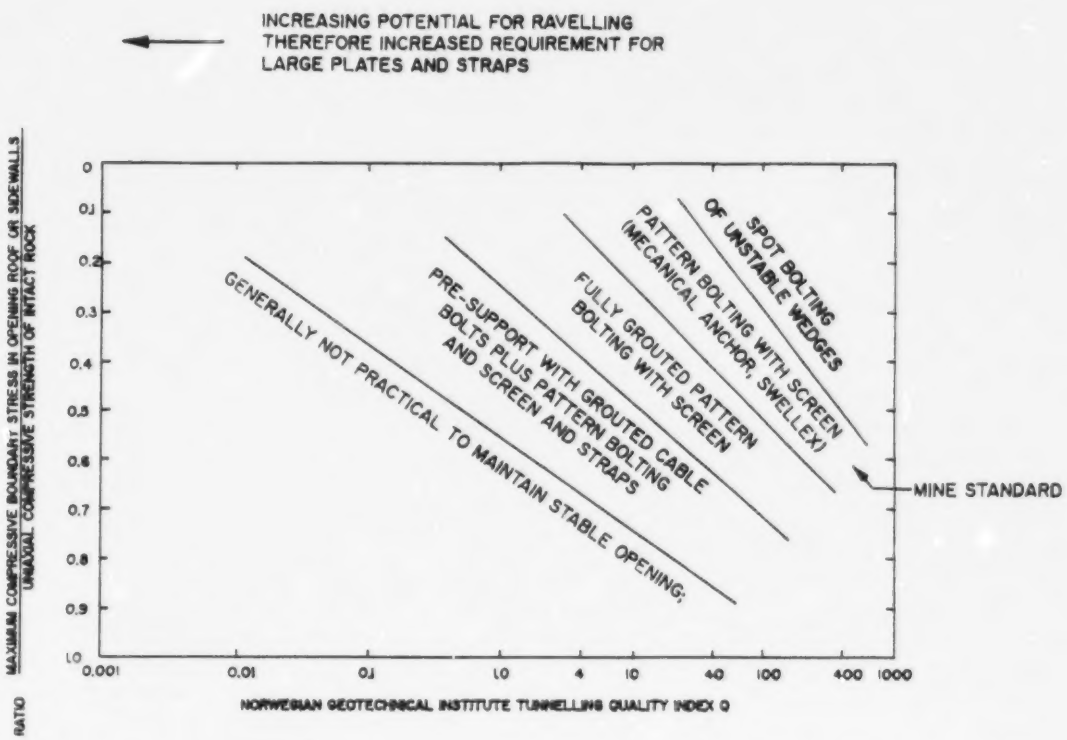
EMPIRICAL SUPPORT DESIGN CHART FOR THOMPSON MINE

Figure 5-24



$$\text{ROCK MASS QUALITY, } Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

1 For excavations up to 4.5 m (15 ft.) span: (barton, 1974)
Minimum bolt length = 2 + 0.05 (span in m) for back.
Minimum bolt length = 2 + 0.05 (height in m) for walls.
Maximum bolts spacing = 1/2 x bolt length.

2 For excavations greater than 4.5 m (15 ft.) span refer to cable bolt design charts, Figures 5-5 and 5-6.

Reference: Hoek, 1980, and Golder Associates

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APPENDIX 5-A

DESCRIPTIONS AND SPECIFICATIONS FOR GROUND SUPPORT SYSTEMS

1

MECHANICALLY ANCHORED, TENSIONED ROCKBOLTS

Background:

Mechanically anchored rockbolts are the most common and probably the oldest form of support in underground mining excavations. An anchor is used to secure the rockbolt at the toe of a drill hole while a face plate holds the front. Figure 1 illustrates a number of components which can be used in different combinations. The expansion shell anchor, shown, is one of a large number of different types, all of which operate in a similar manner; a wedge attached to a bolt shank is pulled into a conical anchor shell, forcing it to expand against the drillhole walls. Provided that the rock is of sufficient quality for the anchor, the rock bolt can be tensioned to its maximum load carrying capacity.

The anchor is usually the weakest link. The bolt's capacity is dependent upon the anchor type and rock mass properties. The anchor is often seen to slip with time, releasing tension. In good quality rock, with a correctly seated anchor, failure will be in the threads holding the face plate or anchor.

After tensioning the bolt it is possible to grout it in place. When grouting a grout seal should be used to center the bolt and seal the collar. Grouting can be conducted at a time after initial installation and the occurrence of significant dilation. Grouting also provides some protection against corrosion.

Ungrouted the bolt will be uniformly stressed over its entire length. Failure of the bolt at any point causes total loss of support. Where unravelling occurs at the collar, causing loss of support a larger face plate may remedy the situation. Wooden or domed plates should not be used as they result in loss of tension.

Tensioned rockbolts are very effective in retaining loose rock about the periphery of an excavation. This also conserves the inherent strength of the rockmass. The support must be capable of supporting the dead weight of any loosened material. In most cases loosening does not penetrate far into the rockmass.

Mechanically anchored tensioned rockbolts are often coupled with straps or mesh to provide an effective support system for a variety of conditions. The use of grout increases their range of application even further.

Advantages:

Can be tensioned immediately after installation, typically to 70% ultimate strength.
Can be grouted at a later stage when short term movements have ceased.
Inexpensive
Quick installation.

Disadvantages:

Subject to corrosions unless grouted.
Anchor is typically weakest link - capacity is dependent upon anchor type and rock mass properties.
Anchor is often observed to slip over time, particularly if experiencing dynamic loading.
If collar plate breaks, or there is excessive anchor slip support capacity is reduced to zero.
Correct installation requires skilled workmen.

Applications:

Grouted mechanically anchored bolts are widely used for permanent support applications in civil engineering. Mechanically anchored bolts without grout are widely used in mining, due to anchor slip. They are not recommended for use close to blasting operations. Ungrouted, these bolts are most efficient where discontinuities have rough and undulating surfaces, friction over these surfaces is improved by the addition of normal forces.

Provides reliable anchorage in good rock, i.e., compressive strength greater than 40 MPa, discontinuity spacing greater than 0.3 m.

Typical Data:

Steel designation	ASTM A 615 (Grade 60) or ASTM A 722 (Grade 150)
Yield stress	414 MPa (60000 psi) or 1034 MPa (150000 psi)
Steel diameter	19 mm (3/4") to 35 mm (1 3/8")
Hole diameter	41 mm (1 5/8") to 63 mm (2 1/2")
Lengths	variable

3

GROUTED DOWEL

Background:

Grouted dowels were developed as an inexpensive method of support where the use of mechanically anchored bolts was not appropriate and tensioning was not necessary. After placing the dowel in the hole a thick grout is pumped in. Alternatively resin or cement cartridges are used. However, cartridge grout is not recommended for use in ground containing open fractures where loss of grout can render bolts useless. A face plate is usually attached at the collar. This can be tightened to provide some tensioning at the immediate face. A schematic diagram of these bolts is given as Figure 2.

By utilizing a fast setting resin cartridge at the toe with slower setting cartridges along the remainder of the hole it is possible to tension these dowels. However, this procedure is difficult and requires skilled operators.

Untensioned, the dowels should be installed as close as practical to the advancing excavation face. This ensures that relatively little deformation occurs prior to installation, and the inherent strength of the rockmass is maintained.

Stress concentrations in grouted dowels are greatest where the bolt intersects dilating fractures. If the strain exceeds the capacity of the dowel there will be abrupt rupture or breakdown between the element and the grout. If slip occurs or the face plate breaks off, the remaining length of the dowel will continue to provide support.

The ultimate strength of the grout bond is dependent upon grout type, rock strength, hole condition and hole size. Strength and stiffness decrease with increasing hole to dowel diameter ratio. Under high stress or dynamic loading bond strength should be reduced to allow slipping rather than rupture.

The ease of installation of untensioned grouted dowels has resulted in wide spread use in many situations. The bolts are often combined with mesh to form a combined support system. Recent developments include hollow dowels and the incorporation of drill bits. Hollow dowels result in better pumped grout distribution while dowels with drill bits can be bored rather than grouted in place. These are suited for poor ground where the drill holes tend to collapse after the drill steel is withdrawn but prior to bolt installation.

Advantages:

Simple to install.
High corrosion resistance.
Anchored over entire length.
Anchor slip and ravelling at collar do not cause loss of support.
Immediate strength available with fast setting resins.

4

Disadvantages:

Difficult to tension, hence, should be installed prior to significant rock mass deformation occurring.
Grout properties depend on preparation.
May require peripheral equipment (grout pump).

Applications:

Widely used in the mining industry for general support duties. Suitable for use in strong or weak rocks and in highly discontinuous rock. Where the shear resistance of the bolts adds to that of the fractures used widely for permanent excavations as most likely to ensure long term stability.

Typical Data:

Steel designation	ASTM A 615 (Grade 60)
Yield stress	414 MPa (60 ksi)
Steel diameter	19 mm (3/4") to 25 mm (1")
Hole diameter	35-38 mm (1 3/8" to 1 1/2")
Lengths	variable

Pull out strength of 150 to 200 kN/m in hard rock.

5

FRICTION ANCHORED ROCKBOLT - SWELLEX (patented by Atlas Copco)

Background:

Developed by Atlas Copco AB, the Swellex deformed tube is expanded by water pressure to the shape of the drill hole. Mechanical interlock of the bolt and the rock then prevents the bolt from sliding. Expansion of the deformed tube leads to some reduction in length, which puts the Swellex into tension. The system is shown in Figure 3.

The bolts come in several varieties including high strength, high deformation and corrosion resistant.

If the anchor slips or the bolt breaks, support capacity is still present in the remaining bolt. The pull out strength is dependent upon rock type, hole diameter and inflation pressure. In hard rock 100 to 200 kN/m can be expected. This strength is achieved immediately after installation. Sufficient bolt length must be anchored above any material where dead weight is to be supported.

Some tension is developed during installation, however, in order to maintain block interlocking and the inherent strength of the rockmass the bolts should be installed as close to the advancing face as practical.

Advantages:

Rapid and simple installation even with long bolts.
Immediate support after installation.
Installation causes constriction of bolt which tensions face plate against rock surface.

Disadvantages:

Subject to corrosion.
Requires peripheral equipment for installation (pump and wand).

Applications:

Swellex bolts are commonly used in the mining industry for medium-term support requirements. Becoming increasingly used in civil engineering tunnelling work. Suitable for use in weak rock, hard rock and highly discontinuous rock.

Typical Data:

Yield load	130 kN (13 ton f)
Tube diameter	26 mm (1")
Hole diameter	33 mm (1 3/8") to 39 mm (1 1/2")
Lengths	up to 8 m (24 ft.)
Inflations pressure	20 MPa (3000 psi)

6

FRICTION ANCHOR OR "SPLIT SET" (patented by Ingersoll-Rand)

Background:

Developed by Scott in conjunction with the Ingersoll-Rand Company in the U.S.A., this device has gained considerable popularity in the mining industry. As the split tube is forced into a drillhole, the spring action of the compressed tube applies a radial force against the rock and generates a frictional resistance to sliding of the rock on the steel. This frictional resistance increases as the outer surface of the tube rusts. The bolt is shown in Figure 4.

If anchor slip or bolt break occurs the remaining length of dowel continues to provide support. The ultimate pull out strength is dependent upon hole diameter, hole condition and roughness of bolt. In hard rock 30 kN/m can be expected. Pull out resistance increases with time. Sufficient bolt length must be anchored in competent rock to ensure the stability of any material whose dead weight is designed to be supported.

In order to maintain block interlocking and the inherent strength of the rockmass the bolts should be installed as close to the advancing face as is practical.

Advantages:

Simple and quick to install.
Yields rather than fails, hence, useful in moving and bursting ground.

Disadvantages:

Difficult to install lengths greater than 6 ft.
Cannot be tensioned hence must be installed prior to significant rock mass deformation occurring.
Drill hole diameter is critical.
Cannot be grouted.
Subject to corrosion.
Low early strength.

Applications:

Used for relatively light support duties in the mining industry, particularly where short term support is required. Little application in civil engineering at present. Suitable for use in weak and hard rock and highly discontinuous rock. Due to low pull out strength bolt should not be used to suspend blocks, only to pit blocks in place.

Pull out strength is dependent upon hole diameter, smoothness of hole, roughness of bolt. In hard rock 30 kN/m can be expected. Pull out resistance increases with time.

Typical Data:

Yield load	90 kN
Tube diameter	33 mm (1.32"), 39 mm (1.55") and 46 mm (1.81")
Hole diameter	32 mm (1 1/4"), 35 mm (1 3/8"), 41 mm (1 5/8")
Lengths	0.9-3.6 m (3-12 ft.)

7

GROUTED CABLE BOLT

Background:

Grouted cables were introduced to mining for reinforcement of the backs of cut and fill stopes. Cable reinforcement, using tensioned or untensioned, fully grouted cables, is widely used in mining applications. Cables can be installed effectively in very narrow tunnels. They are inexpensive and have a very high load bearing capacity. An illustrative diagram is shown in Figure 5. Cables can be used to replace dowels in most applications.

If bolt break occurs, the remaining length of cable continues to provide support. Bolts should be anchored in competent rock to ensure the stability of any material whose dead weight is to be supported. Anchoring lengths of 16 to 30 times the cable diameter are usually considered sufficient. Bond strength is dependent upon rock conditions hole: cable diameter ratio and grout type. The strength can be increased by the use of buttons and "bird-caged" cable. Plates, straps and mesh can be attached to the cables for additional support of the immediate back.

If the bolts are not to be tensioned, they should be installed prior to ground relaxation. This is often achieved by preplacing the cables, i.e., in cut and fill mining long cables are placed in the back. The cables form a pre-supported back for the subsequent lifts. Support forces are developed as progressive cuts are taken. Bond strength has been observed to decrease where peak particle velocities exceed 500 mm/s close to the cables (Stillborg 1984). As such, some caution must be exercised. Support of shear zones can be achieved from development headings well in advance of mining.

Advantages:

Inexpensive.
Can be installed to any length in small excavations.
Can achieve high bolt loads under various rock conditions.
High corrosion resistance.
Pre-supports backs in subsequent mining lifts.

Disadvantages:

Cable tensioning is difficult.
Cement grout requires several days curing before cable can be loaded.
Use of accelerators results in higher early strength but reduced late strength.
Correct installation requires skilled workmen.
Cable must be clean and free of grease to ensure proper bonding.
Can not be used where continuous water flow occurs.

Applications.

Bolts should be installed prior to ground relaxation if they are not to be tensioned. However, cable bolts are not suitable where deformations are such that failure will occur. Bolts must be anchored in competent rock. Bond strength improves with decreasing grout water/cement ratio; however, this in turn is limited by the capacity to pump the grout. Recent developments include utilizing cables capable of sustaining higher strain.

Tensioned cable bolts are well accepted in civil excavations. The system is gaining increasing use in mining applications. Short cables can replace grouted rockbolts.

8

Typical Data:	
Yield stress	1770 MPa (257000 psi)
Yield load	500 kN (50 ton f)
Cable diameter	20 mm (25/32")
Hole diameter	35 mm (1 3/8")
Length	Any length required.

Ultimate strength of support is governed by in situ conditions and installation practices.

9

MESH

Background:

Mesh is used in circumstances where there is a possibility of small blocks falling out between rockbolt face plates. Two types of mesh are in common use: chainlink and weld mesh. Both are installed tight to the excavation surface using plates on rockbolts for anchoring, see Figure 6.

Chainlink mesh conforms well to any excavation shape; however, it is more difficult to handle than weld mesh. Weld mesh can be constructed of higher strength wire than chainlink. The difficulty of eliminating air pockets trapped behind the links make the use of chainlink incompatible with shotcreting. Both types of mesh are subject to damage by blasting.

Advantages:

Inexpensive for protection provided.
Easy to handle and install.

Disadvantages:

Cannot carry excess load of broken rock.
Easily damaged by flyrock or careless equipment operation.
Difficult to install such that it applies any force to the excavation surface.

Applications:

Mesh is ideal intermediate protection from small pieces of broken rock. Weld mesh was used traditionally as reinforcement for shotcrete, but is rapidly being replaced by steel fibre reinforced shotcrete. Mesh, like straps, is held in place with additional faceplates or washers and nuts on rockbolts, or using separate pins, or with cable slings. Commonly used in drawpoints and extraction drifts.

10

STRAP

Background:

This system is used extensively in mining and civil engineering applications to hold and prevent loosening of slabby ground between rockbolts where most weakness places run in one direction. A schematic diagram is given as Figure 7.

Advantages:

Easy to install.
Very effective in stratified, slabby ground.

Disadvantage:

Cannot be used to control ravelling in loose, blocky ground if the block size is small.

Applications:

Steel straps are used to give roof control in ground where rockbolts or dowels alone cannot hold slabs of rock. Where the weakness planes run in one direction the strength of the rockmass is much higher in the direction of the weakness planes than it is across these planes. Under these circumstances, straps may be a more effective support than mesh. The strap is placed to follow the rock surface and held with regularly placed rockbolts. Straps should always be installed across weakness planes, as straps placed parallel are ineffective.

11

SHOTCRETE

Background:

Pneumatically applied mortar and concrete (shotcrete) are being increasingly used in the support of underground excavations both in civil and mining applications, see Figure 8. Shotcrete is applied either as a dry mix where water is added at the spray nozzle, or wet mix, where the water is added prior to pumping. Flexibility of equipment, batching, and availability favour the dry-mix process.

Shotcrete acts in a similar manner to mesh in that it prevents small pieces of rock form loosening thus retaining the self supporting characteristics of the rockmass. Shotcrete is generally stronger than mesh. Material and equipment have been developed to a high degree of reliability; however, the quality of the installed shotcrete is very dependent upon the equipment operators.

In the past shotcrete was coupled with mesh to produce a stiffer support. More recently steel fibre and micro silica have been added to the mix. This increases residual load bearing capacity and does away with the need for mesh.

Advantages:

Protects against weathering and desiccation.
Provides immediate corrosion resistant support to loosened rock.
Low initial strength is unlikely to result in excessive loading.

Disadvantages:

Correct application requires skilled workmen.
Initial strength is low, requires up to 4 weeks to develop full strength.

Applications:

The system has rapid support potential for loosened rock and changing stress conditions. It is as durable as concrete in most applications. Shotcrete is used extensively in civil applications and mine openings that are semi-permanent to permanent. Shotcrete is also used in situations where ground conditions deteriorate before or during installation of other support types.

STEEL ARCHES

Background:

Steel arches developed from, and mostly replace now, the use of timber supports. The arches are manufactured as either rigid or yielding. They are employed as long term, permanent rock mass support, especially in critical areas such as roadway intersections, see Figure 9.

Advantages:

Very effective under high loading.
Can sustain substantial deformation.

Disadvantages:

Expensive.
Time-consuming to install.
Ineffective unless installed properly.
Subject to failure under eccentric loading.
Passive support.
External support: subject to damage.

Applications:

Steel arches are used to support critical excavations in highly stressed ground. The choice of rigid or yielding supports depends upon application and ground conditions. Since they are placed some distance behind the advancing face, most of the short term movement in the rock has already taken place prior to installation; as such the arches carry only the dead weight load of failing rock about the excavation.

Steel sets have only limited application in hard rock mines as support can be achieved more effectively by other methods. The exceptions are faults and shear zones where it is difficult to anchor rock bolts.

13

CABLE SLINGS

Background:

Cable slings are a combination support system. They combine two, commonly used, support types to arrive at a fixture which exhibits many preferential characteristics without all the disadvantages of the individual systems. Many variations of cable slings exist including lacing with eye bolts, anchoring with Swellex and anchoring with Split Sets (the Scott Cable Sling), this is the most common used in North America. See Figure 10.

The Scott cable sling combines cement grouting, high strength cables, Split Set rockbolts and truss-like suspension. Installation involves anchoring a cable in a drill hole by cement cartridge and Split Set. The Split Set provides initial strength as well as tension. The cement ensures long term strength.

Cable slings and lacing have proven to be a very versatile and easily installed support. They are suitable for control of loose rock, shearing, pillar and corner spalling. Lagging or mesh are typically incorporated to secure the area between the cables.

Advantages:

Versatile.
Some strength available immediately.
Can provide extensive coverage with use of mesh.

Disadvantages:

Maximum strength attained only after cement cures.
Attaining suitable tension may require the use of turnbuckles, or installation of wedges.
Difficult installation, expensive.

Applications:

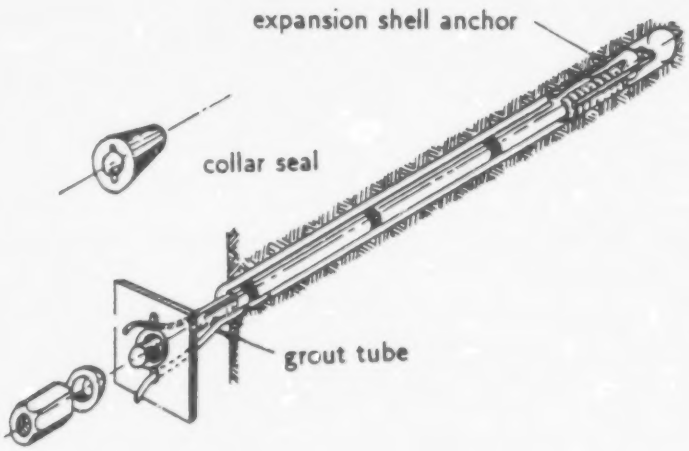
Cable slings and lacing are gaining in popularity in mines as both a "band-aid" type support and as primary support. Versatility makes them suitable to most conditions including high dynamic loading situations. Useful in severe buckling conditions and in highly sheared areas (major shear fault, etc.).

Typical Data:

As per individual components.

MECHANICALLY ANCHORED, TENSIONED ROCKBOLT

Figure 1



Background:

This drawing illustrates a number of components which can be used in different combinations. The expansion shell anchor is one of a large number of different types, all of which operate in basically the same way. A wedge, attached to the bolt shank is pulled into a conical anchor shell forcing it to expand against the drillhole walls. When grouting a bolt, the rubber grout seal is used to centre the bolt in the hole and to seal the collar of the hole against grout leakage. An alternative system is to use a quick setting plaster to seal the hole collar. The grout is injected into the collar end of the hole (except in down-holes) and the return pipe is extended for the length of the hole. Grout injection is stopped when the air has been displaced and grout flows from the return tube.

Advantages:

Bolt can be tensioned immediately after installation using an impact wrench, torque wrench or hydraulic jack, and grouted at a later stage when short term movements have ceased. This system provides very reliable anchorage in good rock and high bolt loads can be achieved.

Disadvantages:

Correct installation requires skilled workmen and close supervision. Grout tubes are often damaged during installation.

Applications:

These bolts are very widely used for permanent support applications in civil engineering. Mechanically anchored bolts without grout are widely used in mining.

Typical data:

Steel designation	ASTM A 615 (Grade 60) or ASTM A 722 (Grade 150)
Yield stress	414 MPa (60000 psi) or 1034 Mpa (150000 psi)
Steel diameter	19 mm ($^3/_4$") to 35 mm ($1\ ^3/_8$")
Hole diameter	41 mm ($1\ ^5/_8$") to 63 mm ($2^1/_2$")
Length	variable

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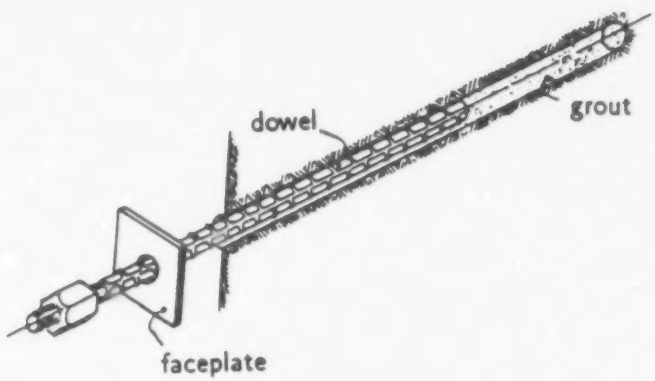
UNTENSIONED GROUTED DOWEL

Figure 2



Background:

Developed as an inexpensive solution where use of untensioned dowels is appropriate. A thick grout is pumped into the drillhole by means of a simple hand pump or a monopump. The dowel is pushed into the grout as shown in the illustration. For up-holes, the dowel is sometimes held in place by bending the dowel slightly during insertion. A faceplate and nut can be added if required although, for light support, a plain dowel is sometimes used. Resin cartridges can also be used to encapsulate the bar and provide bonding to the rock.

Advantages:

Simple and inexpensive. High corrosion resistance in permanent installations.

Disadvantages:

Cannot be tensioned and hence must be installed before significant deformation of the rock mass has taken place. Care must be taken to ensure resin 'grout' is properly mixed. Resin has limited shelf life.

Applications:

Widely used in the mining industry for general support duties.

Typical data:

Steel designation	ASTM A 615 (Grade 60)
Yield stress	414 MPa (60 ksi)
Steel diameter	19mm ($^3/_4$") to 25mm (1")
Hole diameter	35-38mm ($1\ ^3/_8$" to $1\ ^1/_2$")
Lengths	variable

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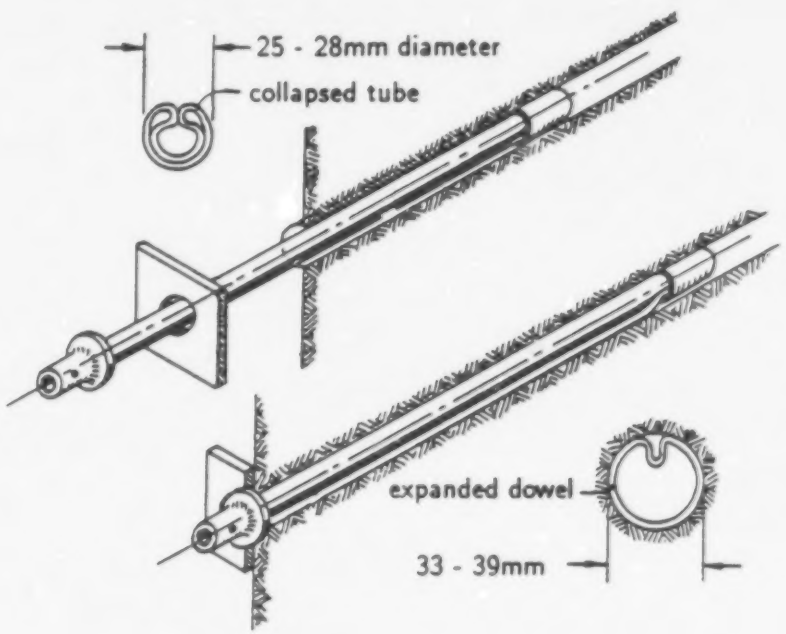
FRICTION ANCHORED ROCKBOLT - SWELLEX

Figure 3



Background:

Developed by Atlas Copco AB, the Swellex deformed tube is expanded by water pressure to the the shape of the drill hole. Mechanical interlock of the bolt and the rock then prevents the bolt from sliding. Expansion of the deformed tube leads to some reduction in length, which puts the Swellex into tension.

Advantages:

Rapid and simple installation. Gives immediate support action after installation. Can be used in a variety of ground conditions. The installation causes contraction in the bolt length. This effectively tensions the face plate against the rock surface.

Disadvantages:

Corrosion can be a problem in long term installations. Requires a pump for installation. May require a sleeve at the collar to prevent spalling under certain rock conditions.

Applications:

Swellex bolts are commonly used in the mining industry for medium-term support requirements. Becoming increasingly used in civil engineering tunnelling work.

Typical data:

Yield load	130 kN (13 ton f)
Tube diameter	26 mm (1")
Hole diameter	33 mm ($1\ ^3/_8$") to 39mm ($1\ ^1/_2$")
Lengths	up to 8m (24 ft)
Inflations pressure	20 MPa (3000 psi)

Warning: Swellex bolts are patented by Atlas Copco AB, Sweden.

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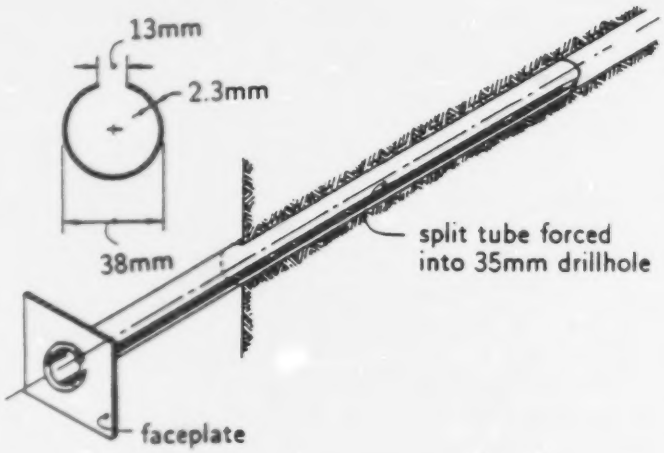
FRICTION ANCHOR OR "SPLIT SET"

Figure 4



Background:

Developed by Scott in conjunction with the Ingersoll -Rand Company in the U.S.A., this device has gained considerable popularity in the mining industry. As the split tube is forced into a drillhole, the spring action of the compressed tube applies a radial force against the rock and generates a frictional resistance to sliding of the rock on the steel. This frictional resistance increases as the outer surface of the tube rusts.

Advantages:

Simple and quick to install and claimed to be cheaper than a grouted dowel of similar capacity. Useful in moving and bursting ground.

Disadvantages:

Cannot be tensioned and hence is activated by movement in the rock in the same way as a grouted dowel. Its support action is similar to that of an untensioned dowel and hence it must be installed very close to the face. The drillhole diameter is critical and most failures during installation occur because the hole is either too small or too large. In some applications, rusting has occurred very rapidly and has proved to be a problem where long term support is required. The device cannot be grouted.

Applications:

Used for relatively light support duties in the mining industry, particularly where short term support is required. Little application in civil engineering at present.

Typical data:

Yield load	90 kN
Tube diameter	33 mm (1.32"), 39 mm (1.55") and 46 mm (1.81")
Hole diameter	32 mm (1 1/4"), 35 mm (1 3/8") 41 mm (1 5/8")
Lengths	0.9-3.6 m (3-1_ ft)

Warning: Split Set friction rock stabilizers are patented by the Ingersoll-Rand Company, U.S.A.

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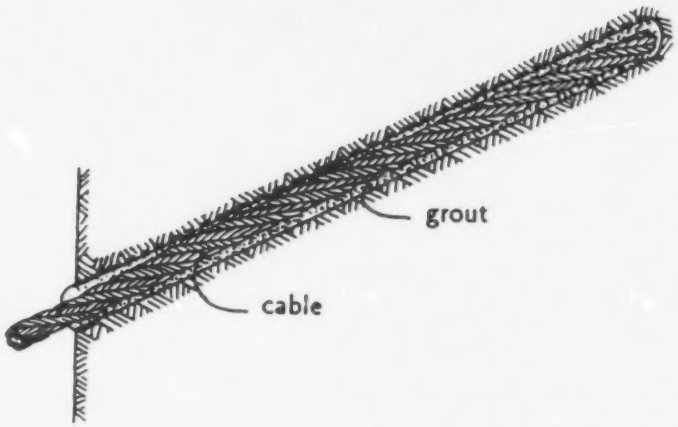
GROUTED CABLE BOLT

Figure 5



Background:

Grouted cables were introduced to mining for reinforcement of the backs of cut and fill stopes. Cable reinforcement, using untensioned or untensioned, fully grouted cables, is very widely used in mining applications. Cables can be installed effectively in very narrow tunnels, they are inexpensive and have a very high load bearing capacity.

Advantages:

This system is inexpensive. If properly installed, it provides competent and durable reinforcement. It can be installed to any length in narrow areas. The system gives very high bolt loads in various rock conditions, as well as high corrosion resistance in permanent installations.

Disadvantages:

Tensioning of the cable bolt is possible only if a special installation procedure is adopted. The use of standard cement in the grout requires several days curing before the cable can be loaded.

Applications:

The system is gaining increasing use in mining applications.

Typical data:

Yield stress	1770 MPa (257000 psi)
Yield load	500 kN (50 ton f)
Cable diameter	20 mm ($^{25}/_{32}$")
Hole diameter	35 mm ($1\,^{3}/_{8}$")
Length	Any length required.

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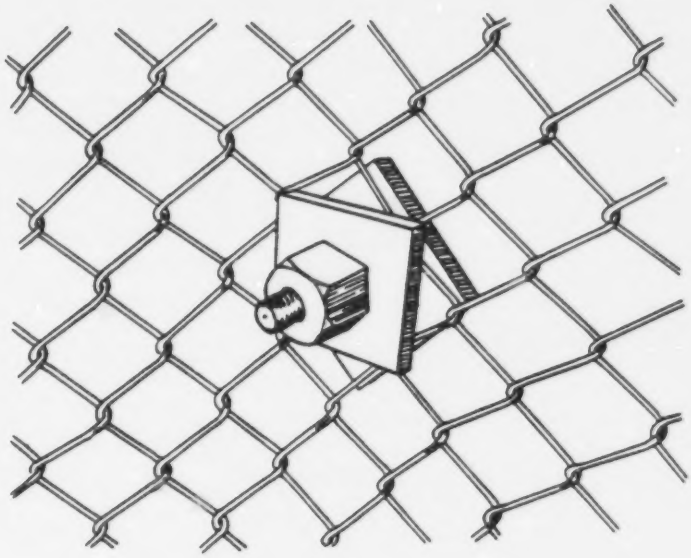
MESH

Figure 6



Background

Wire mesh is used to support small pieces of loose rock and broken rock from falling. Two types of wire mesh currently used in underground operations are chainlink mesh and weld mesh.

Advantages:

Mesh is very economical and quickly installed. It is easy to attach to roof reinforcement with extra faceplates and nuts. It is easily repaired.

Disadvantages:

Mesh cannot carry excessive load of broken rock without failure. It is easily damaged by flyrock from nearby blasts.

Applications:

Mesh is ideal intermediate protection from small pieces of broken rock. Weldmesh is used traditionally as reinforcement for shotcrete, but is rapidly being replaced by steel fibre reinforced shotcrete. Mesh, like straps, is held in place with additional faceplates or washers and nuts on rockbolts or using separate pins.

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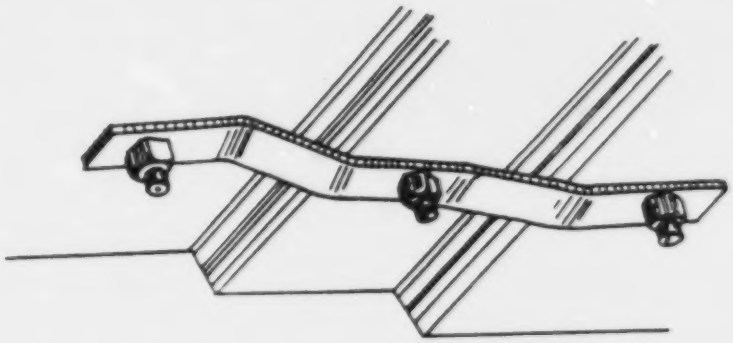
STRAP

Figure 7



Background:

This system is used extensively in mining and civil engineering applications to hold slabby ground between rock bolts or to prevent slabs from loosening.

Advantages:

Straps are easy to install with rock bolts. They are very effective in stratified, slabby ground.

Disadvantages:

Straps cannot be used to control ravelling in loose, blocky ground if the block size is small.

Applications:

Steel straps are used to give roof control in ground where rockbolts or dowels alone cannot hold intermediate and relatively thin slabs of rock. The strap is formed to follow the rock surface and held with regular rock reinforcement or specially installed pins. Straps should always be installed across weakness planes.

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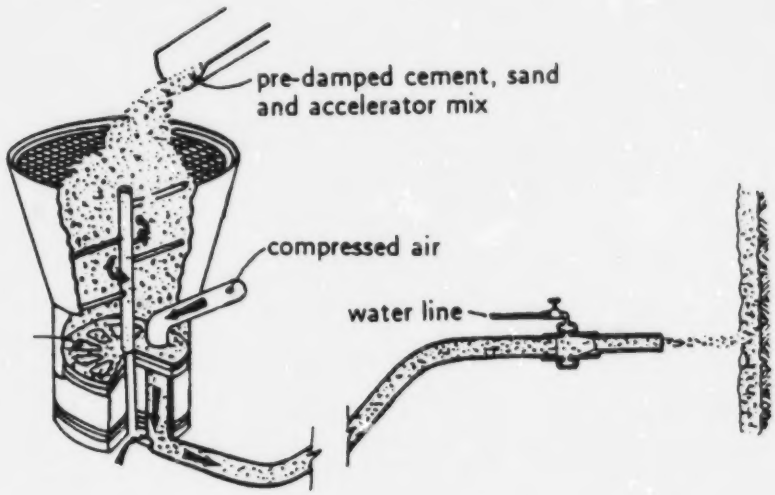
TYPICAL DRY-MIX SHOTCRETE OPERATION

Figure 8



Background:

Pneumatically applied mortar and concrete have been used increasingly in the support of underground excavations both in civil and mining applications. Two basic types of shotcrete (dry-mix and wet-mix) can be used. Flexibility of equipment, batching and availability often enhance the dry-mix process.

Advantages:

This system is very adaptable to the tunnelling environment. New technology has made mix design easier.

Disadvantages:

Final product is highly dependent on the operators and especially on the nozzleman.

Applications:

The system has rapid support potential for loosened rock and changing stress conditions. It is as durable as concrete in most applications for roadways or as roof support. It is not recommended for drawpoints.

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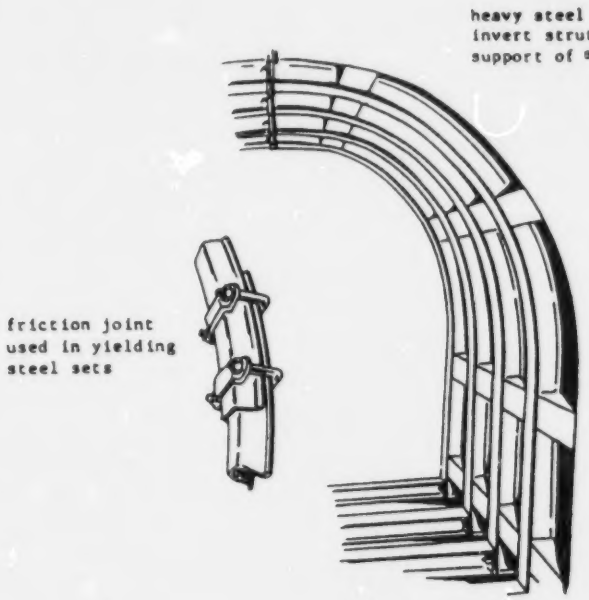
STEEL ARCHES

Figure 9



Background: This system developed from the earliest use of timber supports underground. Steel arches are now manufactured as rigid or yielding supports. They are employed for long-term, permanent rock mass support especially in critical areas of roadways and intersections.

Advantages: Steel arches are highly effective under heavy strata loading conditions and continue to provide support after substantial deformation has occurred.

Disadvantages: Steel arches are time consuming to install properly. Unless well blocked, sets are ineffective. They are prone to twisting under eccentric loading. The rock mass must undergo deformation before an arch starts to take load, i.e. a passive support.

Applications: This system is used in support of long-term haulages and critical intersections especially in highly stressed ground. Yielding and rigid supports are required in different applications.

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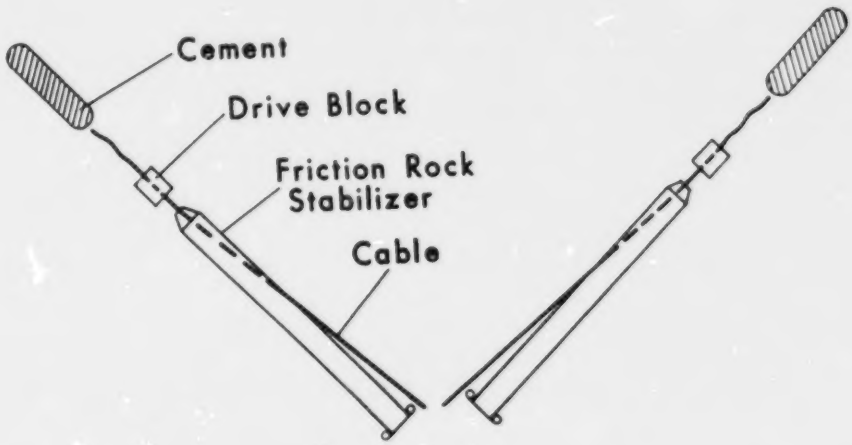
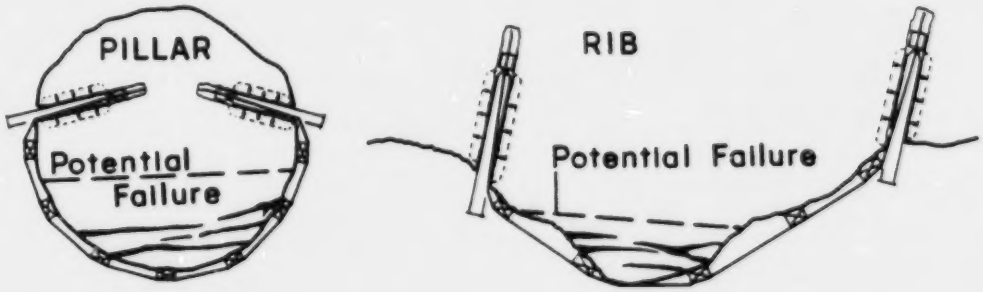
CABLE SLINGS

Figure 10



CABLE SLING COMPONENTS



OTHER CABLE SLING APPLICATIONS

PROJECT NO 852-1605 DRAWN REVIEWED DATE Sept '89

APPENDIX 5-B

GROUND SUPPORT PRACTICES IN CANADA, AUSTRALIA, SWEDEN AND SOUTH AFRICA

TABLE 5B-1:
GROUND SUPPORT PRACTISE IN CANADIAN UNDERGROUND MINES

Company, Mine	Location	Size timbered	Rockbolts & pattern	Grouted anchors	Screening	Shotcrete
Agnico-Eagle, Gold Division	Drifts	8' × 9'	5' & 7' expansion shells as required	⅝" rebar & cement at drawpoints	4" wire mesh 2" link fence as required	
Algoma Steel Corp, MacLeod	various		5' expansion shells & Split Set variable pattern		4" wire mesh as necessary	
Brunswick M&S, No. 12	Drifts	11' × 17' 13' × 18' 14' × 19'	5' expansion shells 4' × 5' 8' Swellex	1" × 8' Dywidag bolts	4" mesh in permanent headings	4" in refuge stations
	Stopes	various	5' expansion shells 4' × 4'	1" × 8' pre-pins 2¼" × 15-60' cable holes, 2 cables per hole	4" × 4" mesh	
Cambior, Yvan Vezina	Drifts & stations		5' expansion shells	⅝" rebar & RocLoc	4" wire mesh	
Canadian Refractories, Kilmar	Drifts	8' × 8' 9' × 9' 10' × 13'	5' & 7' expansion shells 5' Split Set 4' × 4'	⅝" rebar & resins	2" × 2" chain link	
Canamax, Bell Creek	Drifts	9' × 9'	5' expansion shells 4' × 4'			
	Stopes		5' & 8' expansion shells in hanging wall			
Cominco, Polaris	Drifts	5m × 5m	1.8m Split Set 1m × 1m 3.6 & 7.3m Swellex 1.8m × 2m			
Cominco, Potash Ops				3m × 25mm, 2.4m & 1.8m bolts		
Cominco Ltd, Sullivan	Drifts Ramps	9' × 10'	as required 6' Split Set 4' × 4' or as required		4" × 4" welded mesh No. 6 link fence	2" as required as required
Denison Mines Ltd	Drifts & stopes		3-4½-6' expansion shells Swellex 4' × 4'		4" mesh	
Denison-Potacan Potash	All areas		8' expansion shells 5' × 5'	¾" rebar		
Dickenson Mines, AW White	Drifts, raises, ramps & stopes		Expansion shells & Swellex variable lengths & pattern		4" wire mesh	
Dominion Explorers, Lake George	Drifts Stations Stopes	8' × 9' 20' × 40' 275' × 25' × 7'	5' expansions shells 5' bolts & screen 5' bolts 3' × 3'	⅝" rebar as required	2" × 2" galvanized	
Falconbridge, Fraser	Stopes		1.5-2.3m mechanical bolts Cemented cables various patterns		4" 6-gauge wire mesh Expanded mesh as required	
Falconbridge, Lockerby	Prim & secd overcuts & undercuts	12' × 14' 20' cement grouted cables	6' expansion shells	Cement grouted cables on 18' spacing extending 30' beyond the ore outline in the hangingwall and footwall	4" wire mesh	
Falconbridge, Onaping-Craig	Ramps		5' expansion shells 4' × 4'	⅝" × 8-10' cemented rebar	4" No.4 wire	
	Stopes		6' mechanical bolts 4' × 4'	⅝" × 8-10' cemented rebar		

REFERENCE: CANADIAN MINING JOURNAL, 1989.

TABLE 5B-1: CONTINUED

Company, Mine	Location	Size timbered	Rockbolts & pattern	Grouted anchors	Screening	Shotcrete
Falconbridge, Strathcona	Drifts Stopes		5-6' expansion shells 4' × 4' pattern 5-6' expansion shells 4' × 4' pattern	¾" cemented rebar, ¾" cemented cables required	Expanded mesh as required	
Giant Yellowknife, No. 1	Drifts & drawpoints		5' rockbolts 5' Swellex			
Giant Yellowknife, Ross	Drifts		5' expansion shells 4' × 4'			
Giant Yellowknife, Schumacher	Drifts Stations Subleves Raises	occasional	5-6' expansion shells 4' × 4'	20mm × 6' forged head anchors, Cable bolts & FasLoc	4" wire mesh	seldom
Golden Shield, Kerr	Drifts Stopes	8'6" × 8'6" variable	5' & 7' expansion shells	¾" rebar	2" wire mesh with 4" × 4" plates	
Hemlo Gold, Golden Giant	Drifts, ramps, stations, etc		6' & 8' mechanical 8' Swellex 4' × 4'	6' & 8' rebar & Fasloc ¾" cable bolts	2" galvanized chain link mesh	
Hudson Bay, Chisel Lake	Drifts, ramp, etc Stopes Drawpoints		6' expansion shells 4' × 4' 4-5' expansion shells 4' × 4' 6' expansion shells 4' × 4'	¾" rebar & RocLoc		
Hudson Bay, Namew Lake	Drifts & stations		6' expansion shells			
Hudson Bay, Rod	Drifts & stations		6' expansion shells	¾" rebar & FasLoc	4" wire mesh	
Hudson Bay, Ruttan	Drifts Drawpoints		7' expansion shell 4' × 4' 8' Swellex 4' × 4'			
Hudson Bay, South Main shaft	Drifts, stations & stopes		Occasional cable bolt		2" Frost fencing	
Hudson Bay, Spruce Point	Drifts & stations		6' expansion shells 4' × 4'	¾" rebar & FasLoc	4" wire mesh	
Hudson Bay, Stall Lake	Drift, ramp & stopes	7' × 8' 9' × 13'	¾" × 6' bolts 4' × 4'	¾" rebar & FasLoc	2" Frost fence as necessary	
Hudson Bay, Trout Lake	Drifts, stations & stopes		8' expansion shells		2" Frost fencing	
Inco Ltd, Copper Cliff North & South	Drifts & stations		6', 8' & 10' expansion shells & Swellex 4' × 4'	8' × ¾" rebar ¾" cable bolts	4" Nos. 4 & 6 gauge wire mesh	2" where required
Inco Ltd, Crean Hill	Drifts Sills	14' × 13'	6' mechanical 4' × 4' 6' mechanical 4' × 4'	¾" rebar ¾" double cable	4" No. 6 gauge wire mesh 4" No. 6 gauge wire mesh	
Inco Ltd, Frood	Drifts & stations	14' × 12'	6', 8' & 10' expansion shells & Swellex	¾" rebar ¾" cable bolts	4" × 4" Nos. 6 & 9 gauge wire mesh	2" where required
Inco Ltd, Levack	Drifts & stations		6', 8' & 10' expansion shells & Swellex 4' × 4'	¾" rebar & FasLoc ¾" cable bolts	4" × 4" Nos. 6 & 9 gauge wire mesh	2" where required
Inco Ltd, Little Stobie	Drifts, stations & ramps		6' & 8' expansion shells 4' × 4'		4" wire mesh	
Inco Ltd, Stobie	Drifts & VRM sills Drift Drift	12' × 12' 14' × 13' 12' × 14'	6' & 8' expansion shells 4' × 4' Split Sets 4' × 4'	¾" rebar & FasLoc 20' & 30' × ¾" double cable 5' × 7'	4" No. 6 gauge wire mesh 5' × 11'	3" in crusher station 3" in garage
Inco Ltd, Thompson	Drill & mucking sills		8' expansion shells 4' × 4' 6' expansion shells 10' × 10' 6' Split Sets 4' × 4'	20' & 40' cable bolts	4" No. 6 gauge mesh in sheets	2" in drill sills with poor ground

REFERENCE: CANADIAN MINING JOURNAL, 1989.

TABLE 5B-1: CONTINUED

Company, Mine	Location	Size reduced	Rockbolts & pattern	Grouted anchors	Screening	Shotcrete
	Service excavations		(same as above)	Cable slings	4" No. 9 gauge mesh in rises	2" in garages & stations
Kerr Addison Gold Mines	Drifts & ramps		6' x 4'	Rebar & resin	4" No. 9 galv wire mesh	1" wire fibre in exploration & stations
	Stopes		Rockbolts (ore or waste) Swellex (ore) Rebar (waste) 4' x 4' ¾" cablebolts 8' x 8'		none	
LAC Minerals, Bousquet No. 1	Ramps, drifts & ore subs	8" H-beam steel sets	Split Sets, Swellex bolts & cable slings		4" wire mesh	
LAC Minerals, Dovon	Drifts	2.7m x 2.7m	5' expansion shells 5' Swellex		4" wire mesh	
	Ramp	3.6m x 3.6m		¾" rebar & FasLoc	4" wire mesh	
LAC Minerals, Macassa	Drifts	7' x 8'	5' expansion shells 3' x 3'	¾" x 5' rebar & 12" resin cartridges	4" wire mesh	
LynnGold, MacLellan	All headings		2.1m expansion shells 1.2m x 1.2m	22mm rebar & Cemix		
Mines Abcourt	Drifts	14' x 17'	8' expansion shells 4' x 4'	¾" cable	4" wire mesh	
Mines Chibougamau, Copper Rand	Drifts, raises, stopes & stations		3-5-7-10' expansion shells 7' Swellex 4 & 7' Split Set 4' x 4'	¾" rebar & resin 1" grouted Willimas bolts	4" wire mesh	occasional
Mines Chibougamau, Portage	Drifts, raises, stations & stopes	5' x 5' timbered crib	5 & 7' expansion shells Swellex & Split Set 4' x 4'	¾" rebar & resin	3"x"	occasional
Minnova, Ansil	Drifts & ramps		5 & 7' expansion shells 4' x 4'	¾" rebar & FasLoc	4" wire mesh	in garage & crusher station only
Minnova, Opemiska Division	Drifts & stopes		5' & 7' x ¾" bolts 4' x 4'	¾ rebar	2" x 4" & 2" x 2"	
	Sublevel stopes			16' x ¾" grouted cable on 5' centres in hanging wall		
Niobec Inc	Drifts & stations		7' expansion shells 4' x 4'		4" wire mesh	
Noranda, Geco	Drifts & raises		4-6-8' expansion shells 2' x 2' & 4' x 4'	¾" rebar ¾" cablebolts		
Noranda, Matagami Lake		12' x 9' 10' x 13' 10' x 14'	5' & 7' ordinary	¾" grouted cable various lengths	as required	
Noranda, Norita	Drifts		7' expansion shells 4' x 4'	¾" rebar or ¾" cable bolts 8' x 8' pattern	as required	
Placer Dome, Campbell	Drifts & stopes		16' Swellex 4-6-8' mechanical Split Set	¾" rebar cable bolts	2 & 4" wire mesh	2" in stations & cross-cut junctions
Placer Dome, Dome	Drifts	8' x 10'	6' expansion shells 4' x 4' where required	No. 9 rebar & FasLoc as required (rare)	4" wire mesh where required	
	Stopes		6' expansion shells 4' x 4'			
Renabie Gold Mines	Drifts & stations	9' x 9'	6' expansion shells 4' x 4'	4' x ¾" rebar	4" x 6" expanded metal	
Rio Algom, Panel	All locations		6' expansion shells 4' x 4'		2" wire mesh in refuge stations & shops	2" in main shops
Rio Algom, Quirke	All areas		4', 6' & 8' expansion shells 4' x 4' Split sets in walls 6' & 8' Swellex 4' x 4'	1" rebar & Cemix	as required & in service and refuge stations 2" x 2" & 4" x 4" wire mesh	1½ gunnite as required in long term excavations
Rio Algom, Stanleigh	All openings		6' TBE expansion shells 4' x 4' 4' Split Sets on walls	1" rebar & Cemix Swellex as required	4" x 4" No. 6 gauge wire mesh in all openings	1½" in major service openings
SMDC, Star Lake	All areas		6' & 8' expansion shells as required	1" rebar & Cemix H90	4" wire mesh	
Sigma Mines (Quebec)	Drifts & stopes	8' x 9' or 8' x 12' 6-10" posts	5 & 8' expansion shells 3' x 3' 4' x 4' 5' x 5'	1¾" Swellex 20mm rebar & resin	2" x 2" No. 9 gauge wire mesh	
Societe Minere Louvem	Drifts, ramp & stopes		5' random			
Tantalum Mining Corp	Stopes Arched		as required seldom		4" seldom	
Teck Corp, Beaverdell	Drifts	7' x 8' 6" x 7' cap 6" x 7' post	8' expansion shells random			
Teck-Corona, David Bell	Stopes		6' & 8' expansion shells 18' & 25' grouted cables	¾" rebar & RocLoc	4" wire mesh	as required
Teck-Corona, Williams	Drifts & ramps Stopes	4.7m x 3.8m 20m x 25m	1.5m rebar & FasLoc 1.5m Swellex 1.2m x 1.2m	¾" rebar & FasLoc	4" wire mesh	50mm as required
Westmin Resources, HW & Lynx	Drifts & stopes	3.5m x 3.6m to 3.5m x 4.5m	8' expansion & friction	1" x 8' rebar & double strand ¾" grouted cables	4" wire mesh as required	where required
Westroc Industries, Drumbo	All locations		3' Resin 5' x 5'	¾" rebar & FasLoc		

TABLE 5B-2

COMPARISON OF FOUR TYPES OF ROCK BOLTS USED IN CANADIAN SHIELD MINES

Type of bolt	Price * relative figure, approximate)	ease of installation	Quality control on installation needed **	Minimum bonding length required in 6,000 psi (14 MPa) rock ***	Pull-out strength available immediately after installtion	Good resistance to corrosion	Type of ground for which bolt is primarily designed
Mechanically anchored	100	Yes	Yes	Not applicable	Yes	Medium	Hard
Resin-grouted (cartridges)	232	Yes	Less important	0.63 m	Yes	Yes	Weak & hard
Cement-grouted (pumped)	128	No	Less important	0.45 m	No	Yes	Weak & hard
Swellex	237	Yes	Less important	0.5 m	Yes	No	Weak & hard
Split Set	220	Yes	Yes	2.4 m	Partially	No	Weak

* Price relative to mechanical bolt which is given a numerical value of 100

** Because of sensitivity of pull-out strength of the bolt to the borehole diameter tolerance.

*** Failure load / pull-out resistance.

REFERENCE: Choquet 1987.

TABLE 5B-3 GROUND SUPPORT IN AUSTRALIAN MINES

MINE	Fully Bonded				Split Sets	Tensioned Point Anchor			Massive Support		
	Cement			Resin							
	Long Cables	Short Strand	Dowels	Dowels		Slot and Wedge	Expansion Shell	Resin Anchor	Steel Sets	Timber Sets	Shocrete
Warrego					High		High in Past	Few	limited		limited
Mt. Charlott	Moderate	Moderate			Trial only		High				limited
Windarra	Limited				Walls only		High in Past	High	Moderate in Past		High
Hunt					Moderate			Moderate			
ZC/NBHC	Moderate	High						High	Limited	Moderate	Moderate
Cobar	Moderate				Walls only		High	Moderate			
Que River					High		Moderate in Past		Moderate		Limited
Cleveland Tin				Limited			Few				
Rosebury	Limited	Moderate	Moderate			High				Moderate	Limited
Mount Lyell	Moderate	Moderate	Moderate		Few	Moderate			Moderate		Moderate
Mount Isa	Moderate		High	Moderate	Trial Only		Moderate				Limited

High = used extensively as a routine support system.
Moderate = used frequentlyin different routine application.
Few = used infrequently for routine applications,
Limited = used on a non - routine basis over a limited area.

REFERENCE: Fabjancyk, 1982

TABLE 5B-4: GROUND SUPPORT AT MOUNT ISA MINES

		ROCKBOLTS		GROUTED DOWELS					PRESTRESSED GROUTED DOWEL	FRICTION ANCHORS	
		MECH. POINT ANCHORS	RESIN POINT ANCHORS	CARTRIDGE RESIN	CARTRIDGE CEMENT	PUMPED RESIN	PUMPED CEMENT	BLOWN CEMENT	CART RESIN POINT ANCHOR & SLOWSET RESIN	SPLIT SET	SWELLEX
ADEQUATE SUPPORT	GENERAL	Well used proven.	Widely used in 28 mm holes in coal	5000 bolts per period in Isa Mine currently	New product. Not well accepted unproven	Not widely used. Good support. support	Widely used. Good support.	Poor test results Many suspect	Untested but probably fair.	Low strength but excellent yield.	Could replace ALL other systems
	BROKEN GROUND	Suspect, can leaves open & bolt pulls through	Suspect resin can migrate.	Suspect - resin can migrate, undermixing can occur	Grout can migrate.	Good.	Good.	Suspect.	Suspect resin can migrate.	Very good.	Good.
	BLOCKY & WEDGES	Good if anchor secure.	Suspect if resin comes down bolt.	Good if resin does not migrate	OK?	Good.	Good.	Poor	Good if resin does not migrate	Very poor	Good.
	STRESS CHANGES	Good up to anchor secure	Good to anchor strength	Good to anchor strength	OK?	Good.	Good.	Suspect	Good.	Probalby very good.	Good.
CONSISTENT HIGH QUALITY		No due to variable tensio.	No due to resin migrates & trickles.	No due to resin undermix migrate.	Uncertain with cemicron. Fondu can migrate	Good if external mixing OK.	No.	Fair.	Yes.	Yes.	
TIME FOR SUPPORT TO ACT	IMMEDIATE <5 MINS	Support is immediate.	Full strength reached in 5 mins. IMMEDIATE.	Full strength reached in 5 mins. IMMEDIATE	Initial set in 20 mins.	Full strength in 1 hour	High streath in 3 hours.	Slow strength IN gain over 24 hours. TOO SLOW	Full strength in 5 mins. IMMEDIATE	2T/m increasing to 5T/m in a week IMMEDIATE	10T/m available instantly IMMEDIATE
	QUICK <3 HRS				QUICK	QUICK	QUICK				
IMPROVEMENT TO SAFETY AND SAFE TO INSTALL		Yes. Yes.	Yes. Yes.	Yes. Yes.	Yes. Yes.	Yes. Yes.	Yes. Yes.	Current 2 pass system unsafe, and little improvement to safety	Yes. Yes.	Yes. Yes.	Yes. Yes.
SYSTEM MECHANISABLE		Widely used in coal mines, USA.	Semi-mechanical common.	Yes. but messy with many delays.	Cemicron no Fondu Yes.	Yes.	Yes & good	Alimak System.	Yes - Messy? with many delays.	Yes & good	Yes & good
MESHING CAPABILITY		Yes.	Yes.	Yes.	Yes.	Yes.	Yes.	Yes	Yes.	Yes.	Yes.

REFERENCE: MT ISA MINES, 1984.

TABLE 5B-5

EXPERIENCE FROM MOUNT ISA MINES ON PERFORMANCE

SYSTEM	MECH POINT ANCHORS	PUMPED RESIN	PUMPED CEMENT	SPLIT SET	SWELLEX
Life	Short	Long	Long	Short	Medium?
Performance Bolts / Manshift	40	25	25	40	40
Bolts per period Per unit	up to 2000	up to 1000	up to 1000	up to 2000	up to 2000
Installation Loss Rate	Up to 5%	5%	5%	Up to 5%	5%
Meshing Ease Time	Easy Concurrent	Easy Later	Easy Later	Utility bolt Concurrent	Special tool Concurrent
Rebolting	Must rebolt	Must rebolt	Replate only with Dywidag	Unlikely situation	Rebolt?

REFERENCE: Mount Isa Mines, 1984

TABLE 5B-6

SUMMARY OF SWEDISH BOLTING PRACTICE IN DEVELOPMENT OPENINGS

MINE	BOLT TYPE	BOLT LENGTH (m)	BOLT SPACING (m)	COMMENTS
Gaspenberg North	Fully grouted rebar (50%) Swellex (50%)	2.4	1.5	Mesh and shotcrete added as necessary
Kristinberg	Fully grouted rebar	2.3 - 2.7	0.25 - 1.5	Mesh and shotcrete added as required
Langsele	Fully grouted rebar	2.3 - 3.1	0.5 - 1.5	Acid minewater. Mesh and shotcrete added as required.
Malmberget	Fully grouted rebar Resin grouted rebar	2.4	1	Fully grouted rebar being phased out, because of higher
Kiruna	Fully grouted rebar	2.4	1.5 - 2.0	productivity achievements. Some Swellex used in localized areas.

REFERENCE: Aston, 1987

TABLE 5B–7: SUPPORT SELECTION IN SOUTH AFRICAN GOLD MINES

GOOD ROCK MASS CONDITIONS

CASE	PRIMARY SUPPORT (Typical requirements)	SECONDARY SUPPORT (Typical requirements)
Static stress conditions	"Spot" support where necessary (Split Sets, rock studs, etc.)	"Spot" support where necessary (rock studs, etc.)
Stress changes anticipated	"Spot" support where necessary (Split Sets, rock studs, etc.)	Fully grouted tendons > 1,5 m in length, installed on basic 2 m pattern, Support resistance: 30 - 50 kN m2. Rope lacing
Seismic activity anticipated	Split Sets or tendon > 1,2 m in length, installed as close to face as possible, on basic 2 m or 1,5 m	Fully grouted (pref. yield) tendons > 1,5 m in length, installed on basic 2 m pattern. Support resistance: 50 kN/m2. Mesh & lace (hangingwall & sidewalls).

Good rock mass conditions defined as competent rock in shallow to intermediate depth mines, stress or blast induced fracturing slight, unstable rockwall blocks rarely encountered.

AVERAGE ROCK MASS CONDITIONS

CASE	PRIMARY SUPPORT (Typical requirements)	SECONDARY SUPPORT (Typical requirements)
Static Stress Conditions	Rock studs or tendons > 1,5 m integrated with primary support Tendons; or gunite. Support resistance: 30-50 kN/m2.	Steel strapping or rope lacing
Stress changes anticipated	Fully grouted tendons or steel ropes > 1,8 m in length , installed as close to face as possible, on basic 2 m of 1,5 m pattern. Support resistance: 40-60- kN/m2.	Rope lacing and wire mesh integrated with primary support tendons.
Seismic activity anticipated	Fully grouted (pref. yield) tendons or steel ropes, length > 11,8 m, installed as close to face as possible, on a 1,5 m or double 2m pattern. Support resistance: 80-110 kN/m2.	Rope lacing and wire mesh integrated with primary support tendons, plus optional gunite.

Average rock mass conditions defined as competent rock in moderate stress or less competent rock in lower stress conditions. Stress induced fracturing up to 2m deep (or deeper in tunnels larger than 3 x 3 m.) long term deformations of up to 200 mm can occur.

POOR ROCK MASS CONDITIONS

CASE	PRIMARY SUPPORT (Typical requirements)	SECONDARY SUPPORT (Typical requirements)
Static Stress Conditions	(If necessary) shotcrete to face, then fully grouted tendons or steel rope, length > 1,8 m on basic 1,5 m pattern, as close to face as possible. Support resistance: 80-110 kN/m2.	Steel wire mesh integrated with primary support; optional gunite / shotcrete.
Stress changes anticipated	(If necessary) shotcrete to face, then fully grouted steel ropes or Yielding tendons, length > 1,8 m on basic 1 m or double 2 m pattern, as close to face as possible. Support resistance: 120-230 kN/m2.	Rope lacing and wire mesh integrated with primary support. Add integral gunite in long-life tunnels, If neccessary, additional hangingwall support comprising grouted steel ropes.
Seismic activity anticipated	(If necessary) shotcrete to face, then fully grouted steel ropes or yielding tendons, length > 2,3 m on basic 1 m pattern, as close to face as possible. Support resistance: 220-290 kN/m2.	Rope lacing and wire mesh integrated with primary support. add integral gunite in long-life tunnels. If necessary, additonal hangingwall support comprising grouted steel ropes.

Poor, rock mas conditions defined as incompetent rock or high stress or both. Stress induced fracturing to depths of 2 m or more. Long term deformation of more than 200m can occur.

REFERENCE: CMRO, 1988.

APPENDIX 5-C

DATA COLLECTION AND ROCK MASS CLASSIFICATION SYSTEMS

- Barton's Q System
- Laubscher's MRMR System

1

DATA COLLECTION & ROCK MASS CLASSIFICATION

1.0 GENERAL

Planning and design of openings in rock benefit from a number of empirical and semi-empirical rules. These serve to enable the designer to make an estimate of the expected stope dimensions and/or support conditions on the basis of a detailed description of the rock mass. The complete design procedure involves two steps. First, the quality of the rock mass is determined on the basis of a pre-defined classification system, and then the expected performance of the underground opening is predicted using empirically derived correlations with the rock quality factor.

The mining industry generally uses two schemes for classifying rock masses, Laubscher's MRMR and Barton's Q system. Although the systems differ, they rely on similar data to be collected. Once the data is collected, both systems are used to define the quality of the rock mass.

2.0 DATA COLLECTION FROM UNDERGROUND OPENINGS

2.1 Classification Parameters

The following parameters will be defined by data collection.

- Strength of Material;
- Joint Set Number;
- Joint Orientation;
- Joint Spacing;
- Block Shape;
- Joint Surface Alteration;
- Joint Surface Roughness;
- Ground Water;
- Stress Reduction Factor;
- Misc. Information (RQD, Avg. Block Vol.).

2.2 Data Collection Procedure

In order to fill out the data collection form (Figures 1A or 1B), the following procedure for data collection is recommended:

1. Select Area To Be Classified

   Typically this will be the wall or back of a haulage, drill drift or drawpoint. The sampling site should cover an area of approximately 30 m^2 and should consist of predominantly one rock type.

2. Location

   Record location and description of rock. A sketch is often useful for future reference.

3. Strength

The approximate strength (or hardness) of the rock should be determined using the methods described in Table 1.

4. Joint Set Number

From inspection, determine the number of joint sets present. If joint set orientations are reasonably consistent, as is typically the case, this is a straightforward task. Visual recognition of the number of sets should be accompanied by some system of numbering for identification purposes. For example, the most systematic and persistent set can be labelled "Set No. 1" and so on.

The number of sets recorded will tend to be a function of the size of area mapped, and should be interpreted accordingly. The spacing of individual sets will play an important role in this interpretation. For example, four sets recognized following a "conventional" survey of an area (using a stereonet contouring method) may include some sets with such wide spacing that these would be of little relevance to the stability of a short length of drift or stope.

5. Joint Set Orientation

For each of the joint sets defined, measure the mean discontinuity orientation (dip and dip direction). Three to five independent measurements of dip and dip direction should be taken and averaged. Dip should be recorded to the nearest 5 degrees, dip direction to the nearest 10 degrees.

6. Joint Spacing

For each joint set identified, the mean discontinuity spacing or frequency should be determined. This measurement can be rapidly performed as individual joint spacings are not of interest. Five or ten metres can be paced out or estimated with reasonable accuracy (i.e., within 10 per cent of the correct length). The observer should face in the direction of strike for each joint set that is to be counted and count perpendicular to the strike, thereby removing the angular correction factor. Mean spacing or discontinuity frequency over the sampled length can then be quickly determined, along with joint density (joints/m^3).

7. Block Shape

The exposure can be described by the following adjectives to give an impression of block size and shape (see Figure 2 for sketches):

(i) massive - few joints or very wide spacing;
(ii) blocky - approximately equidimensional;
(iii) tabular - one dimension considerably smaller than the other
(iv) columnar - one dimension considerably larger than the other two;
(v) irregular - wide variations of block size and shape;
(vi) crushed - heavily jointed to "sugar cube".

8. Joint Surface Alteration

For each joint set identified, the typical or average alteration conditions should be recorded using the table on the field data collection sheet (Figure 1A) or Table 2 as a guide. This is done in two steps. First, the separation between the joint surfaces is determined and secondly, the appropriate infill description in the table identified.

9. Joint Surface Roughness

For each joint set identified, the typical or average surface roughness should be recorded using the table on the field data collection sheet (Figure 1A) or Table 2 as a guide.

10. Stress Reduction Factor

Where appropriate, the stress reduction factor should be recorded using the table in the data collection sheet (Figure 1A) or Table 3 as a guide. If conditions observed are not found in the table, SRF should be set to one.

11. Other Information

Other information on the form includes average block volume and RQD.

RQD - When no core is available for determining RQD, it is possible to obtain an estimate from the mean discontinuity spacing. This relationship is shown graphically in Figure 4.

3.0 ROCK MASS CLASSIFICATION

3.1 Laubscher MRMR System

Laubscher (1984) defines a Mining Rock Mass Rating (MRMR) obtained from the summation of ratings for intact rock strength 0-20, RQD 0-15, joint spacing 0-25, and joint condition, 0-40, giving a total rating of 0-100. Descriptions and values associated with these basic ratings are given in Figure 3a. Once an overall rating is obtained a series of adjustments are done. These adjustments cater for the effects of:

- weathering;
- orientation of geologic structure in the back and walls of a stope or drift;
- blasting.

The adjustments allow the in-situ rock mass rating to be obtained.

The determination of an overall rating is relatively straightforward using the information given in Figures 3a, 3b, 3c and 3d. The following notes apply.

- Rock Quality Designation (RQD)

The RQD is a quantitative index based on a core recovery procedure in which the core recovery is determined incorporating only those pieces of hard, sound core which are 100 mm or greater in length. Shorter lengths are ignored. RQD is determined from the following expression:

$$RQD(\%) = 100 \times \frac{\text{length of core in pieces 100 mm or longer}}{\text{length of core run}}$$

Core of at least 50 mm in diameter should be used. If core of lesser or greater diameters is to be used, the nominal length of 100 mm should be altered to correspond to two times the core diameter. The International Society of Rock Mechanics recommends that the length of individual core pieces should be assessed along the centre line of the core, so that the discontinuities that happen to parallel the drill hole will not unduly penalize the RQD values of an otherwise massive rock mass. It is important to distinguish between mechanical and natural breaks found in the core. A mechanical break caused by handling should not adversely affect the RQD index which is a measure of the in-situ rock quality. The mechanical broken core segments should be approximated into a solid unit of core in order to arrive at a reliable RQD value.

RQD should be evaluated over an interval not less than 0.5 m. Typically 1.5 to 6 m intervals are used. When no core is available for determining RQD, it is possible to obtain an estimate from the mean discontinuity spacing. This estimate can be obtained from the graph presented in Figure 4. From Figure 3(a), a rating value for the measured RQD is determined.

- Intact Rock Strength (IRS)

The IRS value is the compressive strength of the rock without flaws such as joints and foliation. Unconfined compressive strength tests are usually used to determine this value. However, an estimate of strength derived from hardness is an acceptable starting point (see Table 1). From Figure 3(a), a rating value for the measured strength is determined.

- Joint Spacing

Provision is made in Figure 3(b) for obtaining the joint spacing rating for one, two or three joint systems. In the case of four or more joint sets then the three most frequently occurring joint sets are used (i.e., the closest spaced).

- Joint Condition and Ground Water

The rating is given in Figure 3(c). Consideration is given to discontinuity waviness (Parameter A), roughness (Parameter B), alteration (Parameter C), and infill (Parameter D), under four different ground water conditions.

The condition rating is then a combined percentage of the total possible of 40, that is,

$$\text{Joint condition rating} = 40 \times \frac{A}{100} \times \frac{B}{100} \times \frac{C}{100} \times \frac{D}{100}$$

- Adjustments to MRMR

  - Weathering: for rocks that deteriorate readily during the life of an opening, adjust RQD rating by 95 per cent, UCS rating by 96 per cent, and joint condition by 82 per cent, giving up to 75 per cent adjustment to MRMR.

  - Joint Orientation: the orientation of joint sets will define those rock blocks that may kinematically slide or fall out from an excavation surface. The adjustments, simplified and slightly modified from Laubscher's original proposal are shown in Figure 3d. These are made to the joint spacing rating.

  - Blasting: the excavation method significantly affects the opening periphery. The recommended classification adjustments are as follows:

Boring	100%
Smooth wall blasting	97%
Conventional blasting (good)	94%
Conventional blasting (poor)	80%

3.2 Barton Q System

A similar concept to the MRMR is used in the NGI System (Barton, et al., 1974). In this scheme the structure of the rock mass, the frictional characteristics of fractures, and stresses surrounding the opening are assessed and combined to give the Tunnelling Quality Index (Q).

The system has the form:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

Where:

RQD	= denotes the rock quality designation
J_n	= the joint set number
J_r	= the joint roughness number
J_a	= the joint alteration number
J_w	= the joint water reduction factor
SRF	= the stress reduction factor

The physical significance of the Index is that the term RQD/J_n represents the average block size of the mass; J_r/J_a the average total shearing strength of the joints; J_w both the relative potential head of water on the support and the relative potential of the water eroding the joint infilling; and SRF, both the structurally controlled possibility of dislodgement of blocks of ground and the concentrated stress related confinement, fracturing, squeezing and swelling of the excavation walls.

- Rock Quality Designation (RQD)

The method of determining RQD is discussed in the section on the MRMR classification. The RQD value is used directly in the Q classification system.

- Joint Set Number (J_n)

A rating for the number of joint sets present is determined from the appropriate chart in Table 2.

- Joint Roughness Number (J_r)

A rating for average joint roughness is determined from the appropriate chart on Table 2. Where joint roughness varies the "worst case" surface on a major joint set should be utilized.

- Joint Alteration Number (J_a)

Joint alteration rating is determined from the appropriate chart in Table 2. Both the separation between joint surfaces and joint infilling are considered. Where joint alteration varies, a value corresponding to the average of the "worst case" alteration on a major joint set should be used.

- Joint Water Reduction (J_w)

Due to the extensive and interconnected geometry of underground openings, and the fractured nature of the rock mass immediately adjacent to these openings, it is usually assumed that the rock mass is in a drained condition ($J_w = 1$).

- Stress Reduction Factor (SRF)

The stress reduction factor is determined from the appropriate chart in Table 2. Alternatively the SRF can be determined from relationships relating rock strength and in situ stress, developed by Kirsten (1983). Values can be determined from Table 3.

A significant background of experience has been built up with the NGI system. Hoek and Brown (1980) use the NGI system for evaluation of rock mass strength and Mathews et al (1981) have reported success in using it for assessing open stope design.

In order to relate the index value (Q) to the behaviour and support requirements of an underground excavation an additional quantity called the equivalent dimension (D_e) is used. This dimension is obtained by dividing the span, diameter, or wall height of the excavation by a quantity called the excavation support ratio (ESR).

Hence:

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio}}$$

The excavation support ratio is related to the use for which the excavation is intended and the extent to which some degree of instability is acceptable. Barton (1976) gives the following suggested values for ESR:

	Excavation Category	ESR
A.	Temporary mine openings.	3 - 5
B.	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks) pilot tunnels, drifts and headings for large excavations.	1.6
C.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections.	1.0
E.	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

The ESR is roughly analogous to the inverse of the factor of safety used in the design of rock slopes.

The relationship between the Tunnelling Quality Index Q and the Equivalent Dimension D_e of an excavation which will stand unsupported is illustrated in Figure 5.

4.0 BLOCK GEOMETRY

A useful adjunct to the various classification systems previously discussed is to obtain a measure of block shape. Chappell (1987) has shown that block shape and block volume is important in assessing the re-distribution of stress that takes place when an excavation surface deforms. The initial step in describing block geometry is to determine the average joint spacing or frequency for each of the major joint sets identified. The process of data reduction used to describe joint spacing is outlined below.

The mean spacing Sp for individual joint sets can be obtained by:

$$Sp = 1/\lambda$$

where λ is the mean frequency. Thus, at a specific sampling location, a line of known length should be drawn approximately perpendicular to the strike of the joint set under review. By counting the number of fractures that intersect the line, the fracture frequency and thus average spacing can be estimated.

If it is assumed that joint length is greater than joint spacing, the block size and shape can be defined by either joint spacing or fracture frequency.

For average block size, side lengths L, I and S are defined by mean joint spacing, that is:

L = mean spacing joint set Y = $1/\lambda y$
I = mean spacing joint set X = $1/\lambda x$
S = mean spacing joint set Z = $1/\lambda z$

If four or more joint sets exist, it is assumed that the block is defined by those three sets with the least mean spacing (i.e., the most frequently occurring joints).

Once the edge lengths are obtained, block shape is classified (see Figure 6). The shape is also an indirect measure of anisotropy.

The proposed shape classification system is based on techniques used in sedimentology for the description of grain shape. Folk (1968) defines form as a measure of the relation between the three dimensions of an object. Thus particles (or rock blocks) may be classed qualitatively as compact (or equidimensional) elongated (or rodlike) and platy, with several intermediate categories. The so-called 'form triangle' is shown on Figure 6. The triangle is made up of three axes, namely the ratio of S/L, (L-I)/(L-S) and S^2/LI where S, I, and L are the measured edge lengths of the block. Any two of these axes can be used to define block shape.

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TABLE 1 - FIELD ESTIMATION OF ROCK HARDNESS

Grade	Description	Field Identification	Approximate Range of Uniaxial Compressive Strength (MPa)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025
S2	Soft clay	Easily penetrated several inches by thumb	0.025-0.05
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05-0.10
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10-0.25
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.50
S6	Hard clay	Indented with difficulty by thumbnail	>0.50
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

Reference: Brown, 1981

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX

Description	*Value*	*Notes*
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as $\leq$ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	
G. Three joint sets plus random	12	1. For intersections use ($3.0 \times J_n$)
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	
E. Rough or irregular, planar	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
c. No rock wall contact when sheared.		
H. Zone containing clay minerals thick enough to prevent rock wall contact.	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.	1.0	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
a. Rock wall contact.		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	—

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX (cont'd)

	J_a	ϕ_r (approx.)	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)	
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)	
b. Rock wall contact before 10 cms shear.			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)	
c. No rock wall contact when sheared.			
K. Zones or bands of disintegrated L. or crushed rock and clay (see M. G,H and J for clay conditions)	6.0 8.0 8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
Q. Thick, continuous zones or P. bands of clay (see G, H and R. J for clay conditions)	10.0 - 13.0 13.0 - 20.0	(6° - 24°)	

5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (Kgf/cm^2)	
A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D. Large inflow or high pressure , consioerable outwash of fillings	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10	

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX (cont'd)

6. STRESS REDUCTION FACTOR

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

	SRF
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)	5.0
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)	2.5
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)	5.0
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)	2.5
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)	5.0

b. Competent rock, rock stress problems

	σ_c/σ_1	σ_t/σ_1	SRF
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure

	SRF
N. Mild squeezing rock pressure	5-10
O. Heavy squeezing rock pressure	10-20

d. Swelling rock, chemical swelling activity depending upon presence of water

P. Mild swelling rock pressure	5-10
R. Heavy swelling rock pressure	10-20

1. Reduce these values of SRF by 25 - 50% if the relevent shear zones only influence but do not intersect the excavation.

2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.

3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :
   $RQD = 115 - 3.3J_v$ (approx.) where J_v = total number of joints per m^3
   (RQD = 100 for $J_v < 4.5$)
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX

become the weakest link, and the stability will then depend on the ratio rock-stress/ rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.

5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

TABLE 3 - STRESS REDUCTION FACTOR (SRF)

Material Type	Description of Intact Ground	SRF for Rock Mass Predominated, Fracturing, Squeezing or Swelling																Single Weakness Zone or Joint Infill Predominated Confinement or Squeezing	
		Unconfined Compressive Strength (MPa)	Head of Rock, N, (m)															0	SRF
			2	4	6	8	10	20	40	60	80	100	200	300	400	600	800	1000	0.001	17.5
Granular/ Cohesive Soil	Very Loose/Very Soft	0.035	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	0.002	14.0
	Loose/Soft	0.069	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	0.003	10.3
	Medium Dense/Firm	0.14	11.4	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	0.01	0.2
	Coarse/Stiff	0.28	4.6	11.4	19.5	20	20	20	20	20	20	20	20	20	20	20	20	20	0.04	5.2
	Very Dense/Very Stiff	0.55	1.9	4.7	8.0	11.7	15.7	20	20	20	20	20	20	20	20	20	20	20	0.1	3.9
Rock	Very Soft Rock	1.1	0.8	1.9	3.2	4.7	6.3	15.7	20	20	20	20	20	20	20	20	20	20	0.4	2.4
	Medium Soft Rock	2.2	0.5	0.8	1.3	1.9	2.3	6.3	15.7	20	20	20	20	20	20	20	20	20	1	1.0
	Soft Rock	4.4	0.7	0.5	0.6	0.8	1.1	2.3	6.3	10.7	15.7	20	20	20	20	20	20	20	2	1.4
	Medium Hard Rock	8.8	1.5	0.7	0.5	0.5	0.5	1.1	2.5	4.3	6.3	8.4	20	20	20	20	20	20	3	1.3
	Hard Rock	17.7	3.9	1.5	0.9	0.7	0.6	0.5	1.1	1.7	2.9	3.4	8.4	14.8	20	20	20	20	4	1.1
	Very Hard Rock	35.4	9	3.9	2.2	1.9	1.1	0.6	0.9	0.8	1.1	1.4	3.4	5.7	8.4	14.3	20	20	6	1.0
	Very Very Hard Rock	70.7	5	5	5	3.8	2.8	1.1	0.6	0.3	0.5	0.6	1.4	2.3	3.4	5.7	8.4	11.3	10	0.9
	Extremely Hard Rock	141.3	5	5	5	5	5	2.8	1.1	0.7	0.6	0.5	0.6	1.0	1.4	2.3	3.4	4.5	20	0.7
	Exceptionally Hard Rock	282.4	9	5	5	5	5	5	2.8	1.6	1.1	0.9	0.5	0.5	0.6	1.0	1.4	1.8	30	0.6

ROCK MASS CLASSIFICATION DATA SHEET

Figure **1a**

LOCATION: Level _____ Face _____ Distance _____	DATE

Rock Type	Strength	Sketch	Q =
			MRMR =
			Avg. Vol. =
			Shape
			RQD

Joint Data	Set 1	Set 2	Set 3	Set 4
Dip Direct/Dip Angle				
Average Spacing (ft)				
Estimated No. of Joints/m^3				

JOINT ALTERNATION NUMBER (Ja)

Description of gouge	Joint alteration number (Ja) for joint separation (mm)		
	<1.0[1]	1.0-5.0[2]	>5.0[3]
Tightly healed, hard, nonsoftening impermeable filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	4.0	6.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0[5]	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0[4]	18.0

JOINT ROUGHNESS NUMBER (Jr)

Joint Separation	Condition of Joint	Joint Roughness Number (Jr)
Joints/fissures tight or losing during excavation	Discontinuous joint/fissures	4.0
	Rough or irregular, undulating	3.0
	Smooth undulating	2.0
	Rough or irregular, planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge or sufficient thickness to prevent joint/fissure wall contact upon excavation.	1.0
	Shattered or micro-shattered clays	1.0

JOINT SET NUMBER (Jn)

Number of joint sets	Joint set number (Jn)
Intact, no or few joint/fissures	0.5 - 1.0
One joint/fissure set	2
One joint/fissure set plus random	3
Two joint/fissure sets	4
Two joint/fissure sets plus random	6
Three joint/fissure sets	9
Three joint/fissure sets plus random	12
Four or more joint/fissure sets, random, heavily jointed, sugar cube, etc.	15
Multiple joint/fissure sets (crushed rock, earth-like)	20

STRESS REDUCTION FACTOR (SRF)

Weakness zones intersecting pillar.	10.0
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth).	2.5
Single weakness zones containing clay, or chemically disintegrated rock.	2.5
Multiple shear zones in competent rock (clay free), loose surrounding rock.	7.5
Single shear zones in competent rock (clay free).	2.5
Loose open joints, heavily jointed or 'sugar cube'.	5.0

PROJECT NO ..... DRAWN ..... REVIEWED ..... DATE .....

ROCK MASS CLASSIFICATION

Figure **1b**

Site:	Collected by:	Date:
Description of sample area:		

Rock type	Strength	Sketch

Structural Data	Set 1	Set 2	Set 3	Set 4
Dip/Dip Direction				
Average Spacing (m)				
Estimated No. of Joints/m^3				

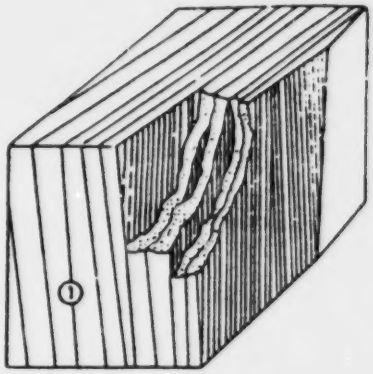
MRMR Classification		Q Classification	
Strength (IRS)		RQD	
RQD		Joint Set No.	
Joint Spacing		Joint Roughness No.	
Joint Condition		Joint Alteration	
Adjustments		Joint Water Reduction	
		Stress Reduction	
MRMR		Tunnelling Quality, Q	

Other Notes:

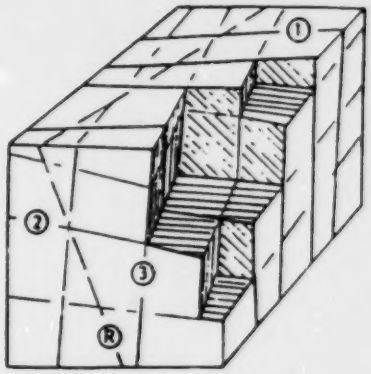
PROJECT NO ..... DRAWN ..... REVIEWED ..... DATE .....

EXAMPLES THAT DEMONSTRATE THE EFFECT OF THE NUMBER OF JOINT SETS ON THE MECHANICAL BEHAVIOUR AND APPEARANCE OF A ROCK MASS

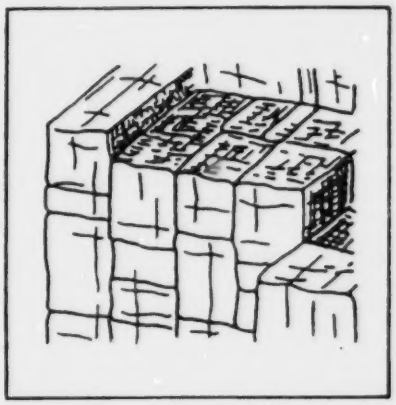
Figure 2



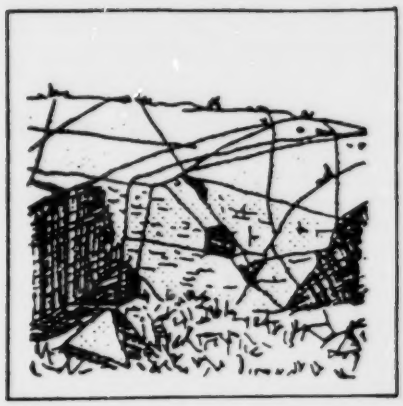
One Joint Set



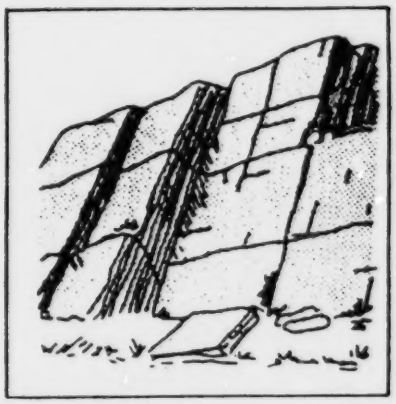
Three Joint Sets



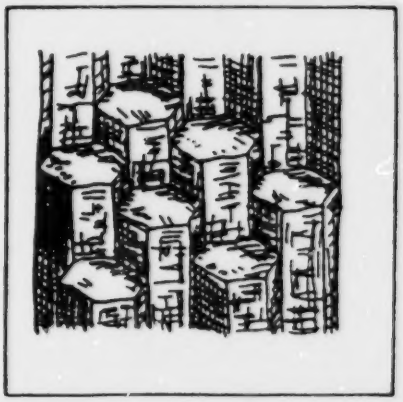
Blocky



Irregular



Tabular



Columnar

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MODIFIED GEOMECHANICS CLASSIFICATION SCHEME (after Laubscher, 1984)

A. MEANING OF THE RATINGS

CLASS	1		2		3		4		5	
	A	B	A	B	A	B	A	B	A	B
RATING(Σ1-4 OF B)	100 - 81		80 - 61		60 - 41		40 - 21		20 - 0	
DESCRIPTION	VERY GOOD		GOOD		FAIR		POOR		VERY POOR	

B. BASIS OF THE CLASSIFICATION

1	ROD %	100 — 97		96-84	83 -71	70 - 56	55 - 44	43 -31	30 - 17	16 - 4	3 — 0	
	RATING (=ROD % x 15/ 100)	15		14	12	10	8	6	4	2	0	
2	IRS (MPa)	>185	184-165	164 145	144 125	124-105	104-85	84-65	64-45	44 - 25	24-5	4-0
	RATING (=0,1=MPa)	20	18	16	14	12	10	8	6	4	2	0
3	JOINT SPACING	REFER TO FIGURE 3b										
	RATING	25 ← 0										
4	JOINT CONDITION INCL. GROUND WATER	REFER TO FIGURE 3c										
	RATING (40xAxBxCxD/ 10^{B})	40 ← 0										

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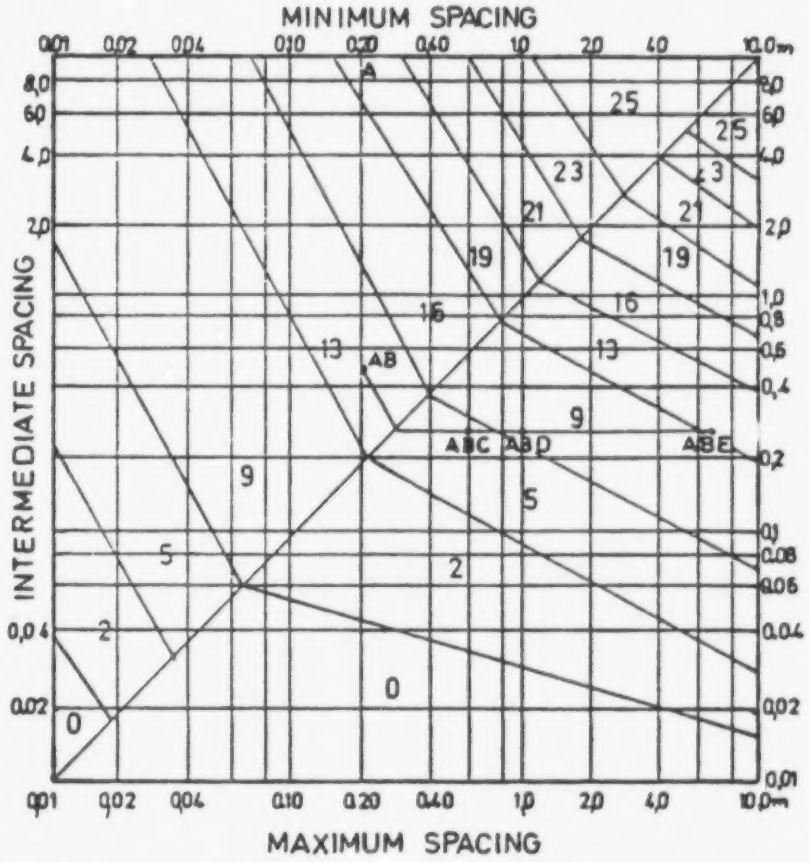
C-19

Figure 3a

JOINT SPACING ADJUSTMENT
(modified from Laubscher, 1984)

Figure **3b**



EXAMPLES:
SPACING A=0.2m, B=0.45m, C=0.5m, D=1.0m, E=7m
RATINGS A=19, AB=13, ABC=5, ABD=9, ABE=13

Locate point corresponding to minimum and intermediate spaced joints (point AB) locate point ABC or ABD by constructing parallel lines read off spacing value.

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ADJUSTMENTS FOR JOINT CONDITION AND GROUNDWATER

Figure 3c

Parameter	Description		Dry condition	Wet Conditions		
				Moist	Moderate pressure 25—125 l/min	Severe pressure > 125 l/min
A Joint expression (large scale irregularities)	Wavy	Multi-directional	100	100	95	90
		Uni-directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint expression (smale scale irregularities or roughness)	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint Wall alteration zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint filling	No fill — surface staining only		100	100	100	100
	Non-softening and sheared material (clay or talc free)	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (eg talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness <amplitude of irregularity		40	30	10	
	Gouge thickness >amplitude of irregularity		20	10	Flowing material 5	

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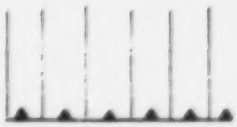
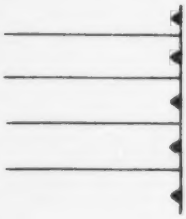
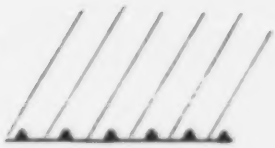
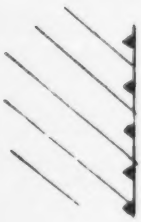
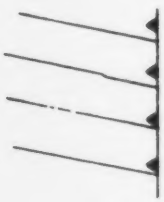
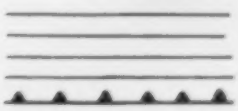
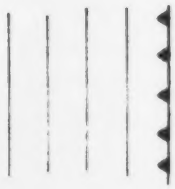
ORIENTATION ADJUSTMENT (MODIFIED FROM LAUBSCHER, 1984)

Figure **3d**

Dip of Structure	Back/Roof	Adjustment	Wall	Adjustment
90°		100%		100%
60°		90%		70%
45°		75%		80%
20°		70%		90%
0		80%		80%

Note: Structure refers to plunge of specific discontinuity set or the plunge of intersection between two major joints sets

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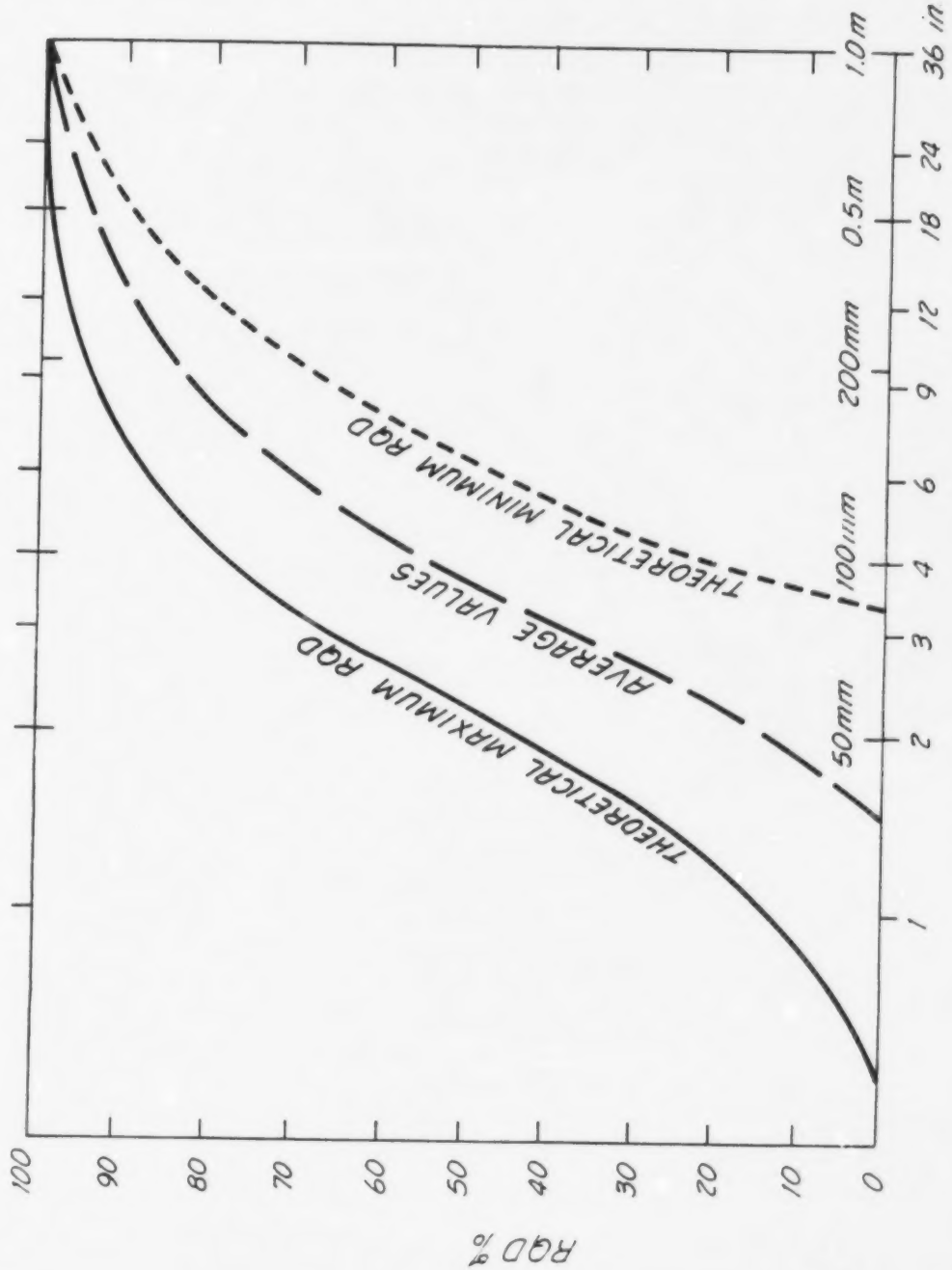
RQD VERSUS MEAN DISCONTINUITY SPACING

Figure 4



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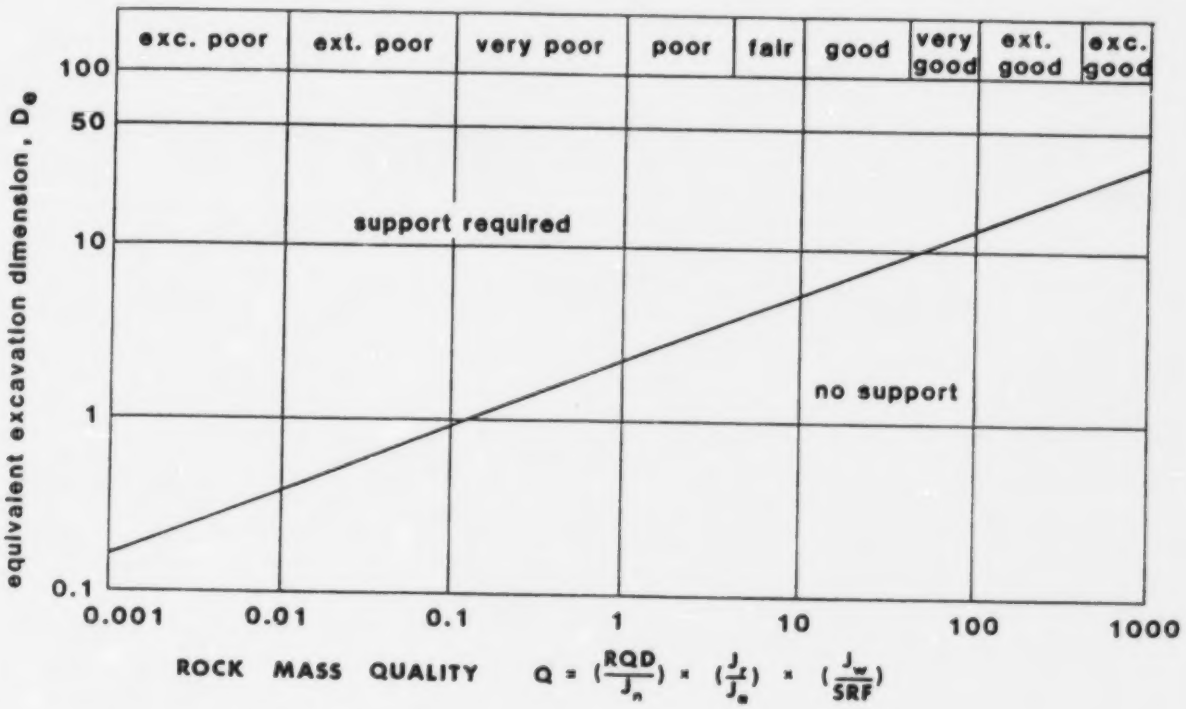
RELATING SPAN AND NGI

Figure 5



from Barton et al, 1974

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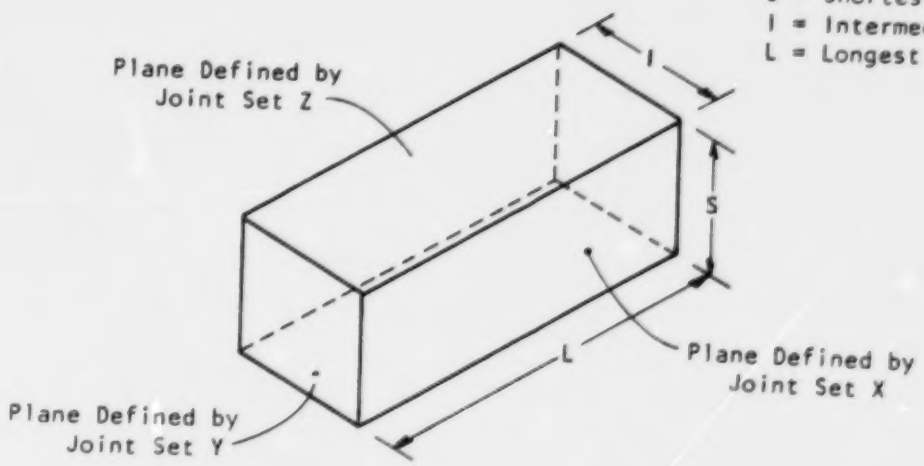
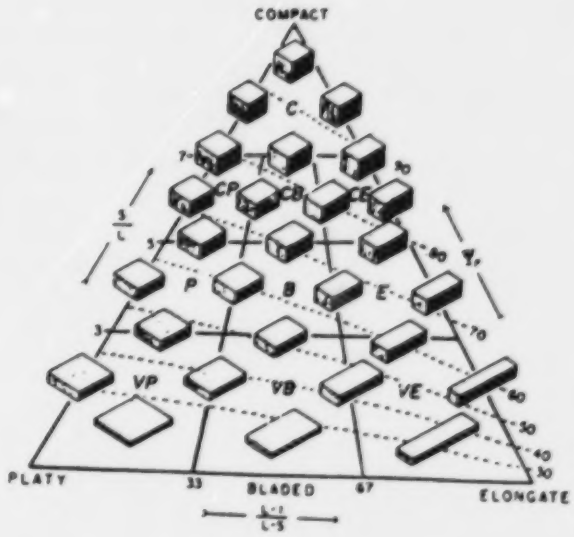
PROPOSED CLASSIFICATION OF BLOCK SHAPE (after Folk, 1965)

Figure 6

S = Shortest Side Length
I = Intermediate Side Length
L = Longest Side Length



Block Shape



C = Compact (cube)
CP = Compact Platy
P = Platy
VP = Very Platy

VB = Very Bladed
B = Bladed
CB = Compact Bladed
CE = Compact Elongate

E = Elongate
VE = Very Elongate

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APPENDIX 5-D

DOCUMENTATION AND PROGRAM LISTING FOR UNDERGROUND WEDGE STABILITY ANALYSIS (UNWEDGE)

D-1
Underground wedge analysis

Introduction

The analysis presented in this appendix was prepared by Dr J.W. Bray of Imperial College in London. Some modifications were carried out by Dr R.D. Hammett of Golder Associates in Vancouver, Canada.

The analysis can be used for wedges defined by three intersecting discontinuities in either the sidewalls or the roof of an underground excavation. For a defined wedge, the analysis determines whether: (a) the wedge floats or falls out, (b) the wedge is inherently stable (the resultant force tending to push it into the rock mass), (c) the wedge tends to slide out on two planes simultaneously, or (d) the wedge tends to slide out on one plane. In the latter two cases, a factor of safety is calculated for given values of the cohesive strength c and the angle of friction ϕ. Rotational instability is not considered.

Note that wedges can always be formed at the wall of an underground excavation, provided that none of the planes 1, 2, 3 and 4 (see diagram on page 518) are parallel, and the discontinuities can be taken to be ubiquitous. This is different from the case of slopes where the existence of wedges has to be established.

The calculation procedure has been specially designed so that the results obtained are independent of whether the normals to the four wedge faces ($\vec{a}$, $\vec{b}$, $\vec{d}$ and $\vec{f}$ in the diagram on page 518) are directed into or out of the wedge, and independent of the order in which the planes 1, 2 and 3 are arranged around the wedge.

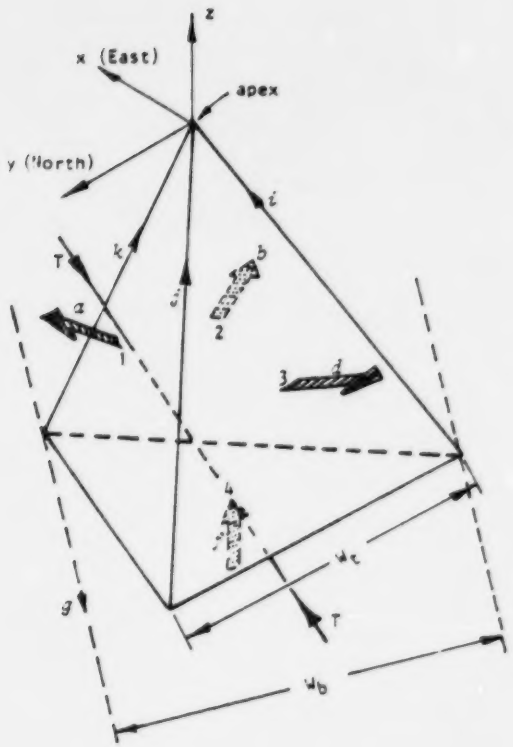
Two solutions are given for wedge sizes defined by

a) the largest wedge that can be released in a tunnel of span or sidewall height W_b, and

b) a wedge defined by the length W_t of the trace formed by the intersection of discontinuity plane 3 and the free face 4.

Notation

1, 2, 3	- discontinuities in the rock mass
4	- free face (roof or sidewall)
W_b	- span of free face between boundary edges
W_t	- length of trace of plane 3 on free face
ψ, α	- dip and dip direction of plane
ψ_5, α_5	- plunge and trend of boundary edges of free face
ψ_6, α_6	- plunge and trend of bolt force T. Note that plunge ψ_6 is defined as *negative* when bolt acts *upwards*.
p_x, p_y, p_z q_x, q_y, q_z t_x, t_y, t_z	- coordinates of vertices of wedge on free face using coordinate system : y axis pointing north, z axis pointing upwards, origin at internal vertex
A_1, A_2, A_3	- areas of faces
W	- weight of wedge



T - bolt force

γ_r - unit weight of rock

u - average water pressure on discontinuities

N, S - normal and shear reactions on plane carried by rock contacts

c, ϕ - cohesion and angle of friction

F_s - factor of safety

Calculation sequence

For wedge defined by span or sidewall height W_b

1. $a_x, a_y, a_z = \text{Sin}\,\psi_1 \text{Sin}\,\alpha_1,\ \text{Sin}\,\psi_1 \text{Cos}\,\alpha_1,\ \text{Cos}\,\psi_1$

2. $b_x, b_y, b_z = \text{Sin}\,\psi_2 \text{Sin}\,\alpha_2,\ \text{Sin}\,\psi_2 \text{Cos}\,\alpha_2,\ \text{Cos}\,\psi_2$

3. $d_x, d_y, d_z = \text{Sin}\,\psi_3 \text{Sin}\,\alpha_3,\ \text{Sin}\,\psi_3 \text{Cos}\,\alpha_3,\ \text{Cos}\,\psi_3$

4. *For overhanging faces (note for vertical faces assume a slight overhang in order to make the dip direction determinate.)*

   $f_x, f_y, f_z = \text{Sin}\,\psi_4 \text{Sin}\,\alpha_4,\ \text{Sin}\,\psi_4 \text{Cos}\,\alpha_4,\ \text{Cos}\,\psi_4$

   For faces with no overhang

   $f_x, f_y, f_z = -\text{Sin}\,\psi_4 \text{Sin}\,\alpha_4,\ -\text{Sin}\,\psi_4 \text{Cos}\,\alpha_4,\ -\text{Cos}\,\psi_4$

5. $g_x, g_y, g_z = \text{Cos}\,\psi_5 \text{Sin}\,\alpha_5,\ \text{Cos}\,\psi_5 \text{Cos}\,\alpha_5,\ -\text{Sin}\,\psi_5$

6. $e_x, e_y, e_z = \text{Cos}\,\psi_6 \text{Sin}\,\alpha_6,\ \text{Cos}\,\psi_6 \text{Cos}\,\alpha_6,\ -\text{Sin}\,\psi_6$

7. $h_x, h_y, h_z = (f_y g_z - f_z g_y), (f_z g_x - f_x g_z), (f_x g_y - f_y g_x)$

8. $i_x, i_y, i_z = (c_y b_z - c_z b_y), (c_z b_x - c_x b_z), (c_x b_y - c_y b_x)$

9. $j_x, j_y, j_z = (a_y c_z - a_z c_y), (a_z c_x - a_x c_z), (a_x c_y - a_y c_x)$

10. $k_x, k_y, k_z = (b_y a_z - b_z a_y), (b_z a_x - b_x a_z), (b_x a_y - b_y a_x)$

11. $l = -(a_x i_x + a_y i_y + a_z i_z)$

12. $l_1 = f_x i_x + f_y i_y + f_z i_z$

13. $l_2 = f_x j_x + f_y j_y + f_z j_z$

14. $l_3 = f_x k_x + f_y k_y + f_z k_z$

15. $m_1 = (h_x i_x + h_y i_y + h_z i_z)/l_1$

16. $m_2 = (h_x j_x + h_y j_y + h_z j_z)/l_2$

17. $m_3 = (h_x k_x + h_y k_y + h_z k_z)/l_3$

18. $e_1 = W_b/|m_1 - m_2|, \quad e_2 = W_b/|m_2 - m_3|, \quad e_3 = W_b/|m_3 - m_1|$

19. e = least of e_1, e_2, e_3

20. $k_1 = -e/l_1, \quad k_2 = -e/l_2, \quad k_3 = -e/l_3$

21. $A_1 = l k_2 k_3/2, \quad A_2 = k_1 A_1/k_2, \quad A_3 = k_1 A_1/k_3$

22. $W = -\gamma_c l A_1 k_1/3$

23. Areas of faces = $|A_1|, |A_2|, |A_3|$

24. Weight of wedge = $|W|$

25. $U_1 = u_1 A_1, \quad U_2 = u_2 A_2, \quad U_3 = u_3 A_3$

26. $r_1 = d_x b_x + d_y b_y + d_z b_z, \qquad t_1 = 1 - r_1^2$

27. $r_2 = a_x d_x + a_y d_y + a_z d_z, \qquad t_2 = 1 - r_2^2$

28. $r_3 = b_x a_x + b_y a_y + b_z a_z, \qquad t_3 = 1 - r_3^2$

29. $T_x, T_y, T_z = T.a_x, T.a_y, T.a_z$

30. $R_x = -U_1 a_x - U_2 b_x - U_3 d_x - T_x.W/|W|$

31. $R_y = -U_1 a_y - U_2 b_y - U_3 d_y - T_y.W/|W|$

32. $R_z = -U_1 a_z - U_2 b_z - U_3 d_z + W - T_z.W/|W|$

33. $g_1 = R_x a_x + R_y a_y + R_z a_z, \qquad h_1 = R_x i_x + R_y i_y + R_z i_z$

34. $g_2 = R_x b_x + R_y b_y + R_z b_z, \qquad h_2 = R_x j_x + R_y j_y + R_z j_z$

35. $g_3 = R_x d_x + R_y d_y + R_z d_z, \qquad h_3 = R_x k_x + R_y k_y + R_z k_z$

36. $N_1 = (g_1 - r_3 g_2) A_1/t_3|A_1|, \qquad N_2 = (g_2 - r_3 g_1) A_2/t_3|A_2|$

37. $S = h_3 A_1 A_2/|A_1 A_2| \sqrt{t_3}$

38. If $N_1 > 0$, $N_2 > 0$ and $S < 0$, the active forces drive the wedge into the rock mass and there is no tendency to fall or slide. The factor of safety has no meaning and the wedge is stable. Terminate computation.

39. If $N_1 > 0$, $N_2 > 0$ and $S > 0$, contact occurs on planes 1 and 2 with separation on plane 3.

Factor of safety $F_s = (N_1 \text{Tan}\,\phi_1 + c_1|A_1| + N_2 \text{Tan}\,\phi_2 + c_2|A_2|)/S$

Terminate computation.

40. $N_2 = (g_2 - r_1 g_3)A_2/t_1|A_2|$, $N_3 = (g_3 - r_1 g_2)A_3/t_1|A_3|$

41. $S = t_1 A_2 A_3/|A_2 A_3| + t_1$

42. If $N_2 > 0$, $N_3 > 0$ and $S < 0$, same conditions apply as for step 38, wedge is stable. Terminate computation.

43. If $N_2 > 0$, $N_3 > 0$ and $S > 0$, contact occurs on planes 2 and 3 with separation on plane 1.

Factor of Safety $F_s = (N_2 \text{Tan}\,\phi_2 + c_2|A_2| + N_3 \text{Tan}\,\phi_3 + c_3|A_3|)/S$

Terminate computation.

44. $N_3 = (g_3 - r_2 g_1)A_3/t_2|A_3|$, $N_1 = (g_1 - r_2 g_3)A_1/t_2|A_1|$

45. $S = t_2 A_3 A_1/|A_3 A_1| + t_2$

46. If $N_3 > 0$, $N_1 > 0$ and $S < 0$, same conditions apply as for step 38, wedge is stable. Terminate computation.

47. If $N_3 > 0$, $N_1 > 0$ and $S > 0$, contact occurs on planes 3 and 1 with separation on plane 2.

Factor of Safety $F_s = (N_3 \text{Tan}\,\phi_3 + c_3|A_3| + N_1 \text{Tan}\,\phi_1 + c_1|A_1|)/S$

Terminate computation.

48. $N_1 = g_1 A_1/|A_1|$

49. If $N_1 < 0$, separation on plane 1, continue with step 53

50. $S_x = R_x - g_1 a_x$, $S_y = R_y - g_1 a_y$, $S_z = R_z - g_1 a_z$

51. $S = (S_x^2 + S_y^2 + S_z^2)^{\frac{1}{2}}$

52. Contact on plane 1, separation on planes 2 and 3,
Factor of Safety $F_s = (N_1 \text{Tan}\,\phi_1 + c_1|A_1|)/S$
Terminate computation.

53. $N_2 = g_2 A_2/|A_2|$

54. If $N_2 < 0$, separation on plane 2, continue with step 58

55. $S_x = R_x - g_2 b_x$, $S_y = R_y - g_2 b_y$, $S_z = R_z - g_2 b_z$

56. $S = (S_x^2 + S_y^2 + S_z^2)^{\frac{1}{2}}$

57. Contact on plane 2, separation on planes 3 and 1.
Factor of Safety $F_s = (N_2 \text{Tan}\,\phi_2 + c_{21}A_2|)/S$
Terminate computation.

58. $N_3 = g_3\, A_3/|A_3|$

59. If $N_3 < 0$, separation on plane 3, wedge is unstable since it falls out, floats out or is forced out by water pressure. Terminate computation.

60. $S_x = R_x - g_3 d_x$, $S_y = R_y - g_3 d_y$, $S_z = R_z - g_3 d_z$

61. $S = (S_x^2 + S_y^2 + S_z^2)^{\frac{1}{2}}$

62. Contact on plane 3, separation on planes 1 and 2,
Factor of Safety $F_s = (N_3 \text{Tan}\,\phi_3 + c_3|A_3|)/S$

For wedge defined by length W_t of trace of plane 3 on free face

Calculation is identical to that described above except for the following steps :

Step 3 - not required

Step 7 - $h'_x , h'_y , h'_z = (f_y d_z - f_z d_y) , (f_z d_x - f_x d_z) , (f_x d_y - f_y d_x)$

$$s = (h'^2_x + h'^2_y + h'^2_z)^{\frac{1}{2}}$$

$$h_x , h_y , h_z = h'_x/s , h'_y/s , h'_z/s$$

Step 17 - not required

Step 18 - not required

Step 19 - $s = W_t/|n_1 - n_2|$

Examples

Problem 1 :

ψ_1	α_1	ψ_2	α_2	ψ_3	α_3	*overhang*	ψ_4	α_4	ψ_5	α_5	ψ_6	α_6
45	240	45	120	45	0	no	90	180	45	90	0	0

c_1	ϕ_1	c_2	ϕ_2	c_3	ϕ_3	γ_r	u	W_b	T
0	30	0	30	0	30	0.027	0	10	0

Output :

$A_1 = 20.41241$, $A_2 = 20.41241$, $A_3 = 40.82483$

$W = 1.53099$

$N_1 = 0.9000$, $N_2 = 0.9000$, $S = 0.71151$

Contact on planes 1 and 2, separation on plane 3

Factor of Safety $F_s = 1.46059$

Problem 2 :

Identical to problem 1 except that overhang exists

Areas and wedge weight identical to problem 1.

N_3 = 1.125 , S = 1.125

Contact on plane 3 , separation on planes 1 and 2

Factor of Safety F_s = 0.57735

The above two cases represent opposite vertical faces of an excavation and show the marked instability of the wedge on one side as compared with the other.

Problem 3 :

ψ_1	α_1	ψ_2	α_2	ψ_3	α_3	*over-hang*	ψ_4	α_4	ψ_5	α_5	ψ_6	α_6
50	290	60	345	35	140	yes	0	180	0	353	0	0

c_1	ϕ_1	c_2	ϕ_2	c_3	ϕ_3	γ_r	u	W_b	T
0	30	0	30	0	30	1	0	4	0

Output:

A_1 = 0.89273 A_2 = 0.93425 A_3 = 2.31098

W = 0.54361

N_1 = -0.34943 N_2 = -0.27181 N_3 = -0.44530

All normal reactions are negative and hence wedge falls out.

Problem 4 :

Identical to problem 3 but with no overhang.

Areas and weight of wedge identical to problem 3.

N_1 = 0.31271 N_2 = 0.05231 S = -0.41476

Wedge is stable under conditions defined in step 38.

The above two cases represent wedges in the roof (problem 3) and floor (problem 4) of an excavation.

```
10    REM "UNWEDGE - underground wedge analysis, E.Hoek, August 1984."
20    CLS:PRINT "UNDERGROUND WEDGE ANALYSIS":PRINT
30    INPUT"Tunnel span (S) or trace length (L) ";R$ :PRINT
40    IF R$ = "S" OR R$ = "s" THEN INPUT "Span Wb : ", W1:GOTO 60
50    INPUT"Trace length Wt : ",W2
60    FOR J = 1 TO 3
70    PRINT"dip of plane";J;:INPUT ":",D(J)
80    PRINT"dip direction of plane";J;:INPUT ":",A(J):NEXT J
90    INPUT"Overhang ? yes (Y) or no (N) : ",S$
100   INPUT"dip of free face : ",D(4)
110   INPUT"dip direction of free face : ",A(4)
120   INPUT"plunge of tunnel axis : ",D(5)
130   INPUT"trend of tunnel axis : ",A(5)
140   INPUT"bolt force T : ",T: IF T=0 THEN C1=0:C2=0:C3=0:GOTO 170
150   INPUT"plunge of bolt : ",D(6)
160   INPUT"trend of bolt : ",A(6)
170   FOR J=1 TO 6: IF D(J) = 90 THEN D(J) = 89.99
180   D(J) =D(J)/57.29578 : A(J) = A(J)/57.29578 : NEXT J
190   INPUT"unit weight of rock : ",G
200   FOR J = 1 TO 3
210   LOCATE 2*J+3,40:PRINT "cohesive strength, plane";J;:INPUT ":",C(J)
220   LOCATE 2*J+4,40:PRINT "friction angle, plane";J;:INPUT ":",F(J)
230   F(J) = F(J)/57.29578 : NEXT J
260   LOCATE 11,40: PRINT "water pressure u";:INPUT ":",U
270   A1=SIN(D(1))*SIN(A(1)):A2=SIN(D(1))*COS(A(1)):A3=COS(D(1))
280   B1=SIN(D(2))*SIN(A(2)):B2=SIN(D(2))*COS(A(2)):B3=COS(D(2))
290   D1=SIN(D(3))*SIN(A(3)):D2=SIN(D(3))*COS(A(3)):D3=COS(D(3))
300   F1=SIN(D(4))*SIN(A(4)):F2=SIN(D(4))*COS(A(4)):F3=COS(D(4))
310   IF LEFT$(S$,1)="N" OR LEFT$(S$,1)="n" THEN F1=-F1:F2=-F2:F3=-F3
320   IF R$="L" OR R$="l" THEN G1=0:G2=0:G3=0:GOTO 340
330   G1=COS(D(5))*SIN(A(5)):G2=COS(D(5))*COS(A(5)):G3=-SIN(D(5))
340   IF T=0 THEN 370
350   C1=COS(D(6))*SIN(A(6)):C2=COS(D(6))*COS(A(6)):C3=-SIN(D(6))
370   IF R$="S" OR R$="s" THEN 380
371   H7=F2*D3-F3*D2:H8=F3*D1-F1*D3:H9=F1*D2-F2*D1
372   S7=SQR(H7^2+H8^2+H9^2):H1=H7/S7:H2=H8/S7:H3=H9/S7:GOTO 390
380   H1=F2*G3-F3*G2:H2=F3*G1-F1*G3:H3=F1*G2-F2*G1
390   I1=D2*B3-D3*B2:I2=D3*B1-D1*B3:I3=D1*B2-D2*B1
400   J1=A2*D3-A3*D2:J2=A3*D1-A1*D3:J3=A1*D2-A2*D1
410   K1=B2*A3-B3*A2:K2=B3*A1-B1*A3:K3=B1*A2-B2*A1
420   L=-(A1*I1+A2*I2+A3*I3):L1=F1*I1+F2*I2+F3*I3
430   L2=F1*J1+F2*J2+F3*J3:L3=F1*K1+F2*K2+F3*K3
```

```
440   M1=(H1*I1+H2*I2+H3*I3)/L1:M2=(H1*J1+H2*J2+H3*J3)/L2
450   IF R$="S" OR R$="s" THEN 460
451   E=W2/(ABS(M1-M2)):GOTO 510
460   M3=(H1*K1+H2*K2+H3*K3)/L3
470   E1=W1/ABS(M1-M2):E2=W1/ABS(M2-M3):E3=W1/ABS(M3-M1)
480   IF E1<=E2 AND E1<=E3 THEN E=E1:GOTO 510
490   IF E2<=E1 AND E2<=E3 THEN E=E2:GOTO 510
500   IF E3<=E1 AND E3<=E2 THEN E=E3
510   K4=-E/L1:K5=-E/L2:K6=-E/L3
520   A4=L*K5*K6/2:A5=K4*A4/K5:A6=K4*A4/K6
530   W=-G*L*A4*K4/3:LOCATE 13,40:PRINT "weight of wedge = ";ABS(W)
540   U1=U*A4:U2=U*A5:U3=U*A6
550   R1=D1*B1+D2*B2+D3*B3:T1=1-R1^2
560   R2=A1*D1+A2*D2+A3*D3:T2=1-R2^2
570   R3=B1*A1+B2*A2+B3*A3:T3=1-R3^2
580   T4=T*C1:T5=T*C2:T6=T*C3
590   R4=-U1*A1-U2*B1-U3*D1-T4*W/ABS(W)
600   R5=-U1*A2-U2*B2-U3*D2-T5*W/ABS(W)
610   R6=-U1*A3-U2*B3-U3*D3+W-T6*W/ABS(W)
620   G4=R4*A1+R5*A2+R6*A3:G5=R4*B1+R5*B2+R6*B3
630   G6=R4*D1+R5*D2+R6*D3:H4=R4*I1+R5*I2+R6*I3
640   H5=R4*J1+R5*J2+R6*J3:H6=R4*K1+R5*K2+R6*K3
650   N1=(G4-R3*G5)*A4/(T3*ABS(A4))
660   N2=(G5-R3*G4)*A5/(T3*ABS(A5))
670   S=(H6*A4*A5)/((ABS(A4*A5))*SQR(T3))
680   IF N1<=0 OR N2<=0 GOTO 760
711   LOCATE 14,40:PRINT "normal force N1 = ";N1
712   LOCATE 15,40:PRINT "normal force N2 = ";N2
713   LOCATE 16,40:PRINT "shear force S = ";S
720   IF S<=0 THEN LOCATE 18,40:PRINT "wedge stable":GOTO 1200
730   F=(N1*TAN(F(1))+C(1)*ABS(A4)+N2*TAN(F(2))+C(2)*ABS(A5))/S
740   LOCATE 18,40:PRINT "sliding on planes 1 and 2"
750   LOCATE 19,40:PRINT"factor of safety = ";F:GOTO 1200
760   N2=(G5-R1*G6)*A5/(T1*ABS(A5)):N3=(G6-R1*G5)*A6/(T1*ABS(A6))
770   S=H4*A5*A6/(ABS(A5*A6)*SQR(T1))
780   IF N2<=0 OR N3<=0 GOTO 830
790   LOCATE 14,40:PRINT "normal force N2 = ";N2
800   LOCATE 15,40:PRINT "normal force N3 = ";N3
801   LOCATE 16,40:PRINT "shear force S = ";S
802   IF S<=0 THEN LOCATE 18,40:PRINT "wedge stable":GOTO 1200
803   F=(N2*TAN(F(2))+C(2)*ABS(A5)+N3*TAN(F(3))+C(3)*ABS(A6))/S
805   LOCATE 18,40:PRINT "sliding on planes 2 and 3"
```

```
806   LOCATE 19,40:PRINT "factor of safety = ";F:GOTO 1200
830   N3=(G6-R2*G4)*A6/(T2*ABS(A6)):N1=(G4-R2*G6)*A4/(T2*ABS(A4))
840   S=H5*A6*A4/(ABS(A6*A4)*SQR(T2))
850   IF N3<=0 OR N1<=0 GOTO 900
851   LOCATE 14,40:PRINT "normal force N1 = ";N1
852   LOCATE 15,40:PRINT "normal force N3 = ";N3
853   LOCATE 16,40:PRINT "shear force S = ";S
860   IF S<=0 THEN LOCATE 18,40:PRINT "wedge stable":GOTO 1200
870   F=(N3*TAN(F(3))+C(3)*ABS(A(6))+N1*TAN(F(1))+C(1)*ABS(A4))/S
880   LOCATE 18,40:PRINT "sliding on planes 1 and 3"
890   LOCATE 19,40:PRINT "factor of safety = ";F:GOTO 1200
900   N1=G4*A4/ABS(A4):IF N1<=0 GOTO 960
910   S4=R4-G4*A1:S5=R5-G4*A2:S6=R6-G4*A3:S=SQR(S4^2+S5^2+S6^2)
920   LOCATE 14,40:PRINT "normal force N1 = ";N1
930   LOCATE 15,40:PRINT "shear force S = ";S
940   F=(N1*TAN(F(1))+C(1)*ABS(A4))/S
941   LOCATE 17,40:PRINT "sliding on plane 1"
942   LOCATE 18,40:PRINT "factor of safety = ";F:GOTO 1200
960   N2=G5*A5/ABS(A5):IF N2<=0 GOTO 1020
970   S4=R4-G5*B1:S5=R5-G5*B2:S6=R6-G5*B3:S=SQR(S4^2+S5^2+S6^2)
980   LOCATE 14,40:PRINT "normal force N2 = ";N2
990   LOCATE 15,40:PRINT "shear force S = ";S
1000  F=(N2*TAN(F(2))+C(2)*ABS(A5))/S
1011  LOCATE 17,40:PRINT "sliding on plane 2"
1012  LOCATE 18,40:PRINT "factor of safety = ";F:GOTO 1200
1020  N3=G6*A6/ABS(A6):IF N3<=0 GOTO 1080
1030  S4=R4-G6*D1:S5=R5-G6*D2:S6=R6-G6*D3:S=SQR(S4^2+S5^2+S6^2)
1040  LOCATE 14,40:PRINT "nornal force N3 = ";N3
1050  LOCATE 15,40:PRINT "shear force S = ";S
1060  F=(N3*TAN(F(3))+C(3)*ABS(A6))/S
1065  LOCATE 17,40:PRINT "sliding on plane 3"
1070  LOCATE 18,40: PRINT "factor of safety = ";F:GOTO 1200
1080  LOCATE 14,40:PRINT "wedge falls out"
1200  IF INKEY$="" THEN 1200 ELSE 20
```

APPENDIX 5-E

DOCUMENTATION AND PROGRAM LISTING FOR VOUSSOIR ARCH STABILITY ANALYSIS (VOUSAN)

PROGRAM : VOUSAN -- Version 1.1

Roof Design in Bedded Rock

(Revised by TPF Golder Associates, May, 1986)

- This program uses a relaxation technique for solving the indeterminate equations of the voussoir beam model of a roof bed.

The three possible failure modes addressed are as follows:

1. Shear at the abutment when the limiting shear, tan(phi), is less than the required abutment vertical reaction.

2. Crushing at the hinges formed in the beam crown and lower abutment contacts.

3. Buckling of the roof beam with increasing eccentricity of lateral thrust, and a consequent tendency to form a 'snap-through` mechanism.

The relaxation technique involves an assumption for the internal load distribution, and iteration to determine the equilibrium state. The beam is assumed initially to be free of lateral thrust, and subject only to gravity loading. From these specified loads, the relaxation process aims to find the final state of lateral thrust. The operational response of the beam can then be determined.

The relaxation procedure starts from an assumption of the load/depth ratio,n, from which can be calculated an initial lever arm ,z, for the abutment and halfspan internal forces, using the equation z=t(1-2n/3).

The various roof beam load and deformation parameters can then be calculated directly, from their respective formulae. The solution procedure involves sequential calculation of Fc,Fav,L,DELL,Z and n . Each sequence of computation produces a new value of n, which is then introduced into the formula for Fc to recommence the solution cycle. Iterations of the solution sequence are continued until stable values of the maximum longitudinal stress (Fc) and the lateral depth ratio (n) are obtained.

Reference: Excavation Design in Stratified Rock Section 8.4 & 8.5 NOTE: "16" has been replaced with "8" in equations 8.10(c) and 8.10(e) in accordance with the correct equation for the arc length of a parabola.

```
c     Variable      -  Definition                                  Units
c     S             -  Span of roof/stope                          m.
c     T             -  Thickness of beam                           m.
c     DIP           -  Dip of ore body                             deg.
c     E             -  Rock Mass Modulus                           MPa
c     UNIWT         -  Unit Weight                                 MN/cu.m.
c     UCS           -  Uniaxial compressive strength               MPa
c     ANTY          -  Type of analysis:  1=Plane Strain
c                                         2=Rectangular Plate
c     AB            -  A/B ratio                                   -
c     PR            -  Poisson ratio                               -
c     PHI           -  rock angle of internal friction             deg.
c
c=====================================================================
c
c                          DIMENSION
c
c=====================================================================

      character sab,cordat
      character*80 proj,stope,date,site

c=====================================================================
c
c                          DATA INPUT
c
c=====================================================================

C     CALL CLS
      WRITE(*,8)
      WRITE(*,*)' Enter the date '
      READ(*,'(a80)')DATE
      WRITE(*,*)' Enter the site.'
      READ(*,'(a80)')SITE
      WRITE(*,*)' Enter the project name and number.'
      READ(*,'(a80)')PROJ
      WRITE(*,*)' Enter the stope identification.'
      READ(*,'(a80)')STOPE

2     WRITE(*,*)' Enter the span of roof/stope in metres.'
      READ(*,*)S
      WRITE(*,*)' Enter the thickness of beam in metres.'
      READ(*,*)T
      WRITE(*,*)' Enter the dip of the orebody.'
      READ(*,*)Dip
      WRITE(*,*)' Enter the rock mass modulus in MPa.'
      READ(*,*)E
      WRITE(*,*)' Enter the unit weight of rock in MN/cu.m..'
      READ(*,*)Uniwt
      WRITE(*,*)' Enter the uniaxial compressive strength in MPa.'
      READ(*,*)UCS
```

```
      WRITE(*,*)' Enter the friction angle in degrees.'
      READ(*,*)PHI
11    WRITE(*,12)
12    FORMAT(   ' Enter conditions:          1=Plane strain'/
     1          '                           2=Rectangular plate')
      READ(*,'(i2)')INTY
      IF ((INTY.NE.1).AND.(INTY.NE.2)) GOTO 11
      IF (INTY.EQ.2) THEN
13    WRITE(*,*)' Enter the POISSON RATIO .'
      READ(*,*)PR
      IF (PR.GT.1.) THEN
C     CALL BEEPIT
C     CALL BEEPIT
      WRITE(*,*)' VALUE MUST BE LESS THAN OR EQUAL 1 !!!!!!!'
      WRITE(*,*)' PLEASE ENTER A NEW VALUE.'
      GOTO 13
      ENDIF
14    WRITE(*,*)' Enter the A/B RATIO.'
      READ(*,*)AB
      IF (AB.GT.1.) THEN
C     CALL BEEPIT
C     CALL BEEPIT
      WRITE(*,*)' VALUE MUST BE LESS THAN OR EQUAL 1 !!!!!!!'
      WRITE(*,*)' PLEASE ENTER A NEW VALUE.'
      GOTO 14
      ENDIF
      ENDIF
```

```
c
c                          VERIFY DATA INPUT
c
```

```
C     CALL CLS
      NPR=0
      WRITE(NPR,8)
      WRITE(*,3)
      WRITE(*,4)date,site,proj,stope
      WRITE(*,5) S,T,Dip,E,Uniwt,UCS,phi
3     FORMAT   (5X,'                                  INPUT DATA'/
     1          ,5X,'                                  ----------'/)
4     FORMAT   (40X,'  DATE          :',A41/
     1          ,40X,'  SITE          :',A41/
     1          ,40X,'  PROJECT       :',A41/
     1          ,40X,'  STOPE         :',A41/)
5     FORMAT   (/, 5X,' Span of roof                  = ',f9.3,' (m.)',
     1/,              5X,' Thickness of beam          = ',f9.3,' (m.)'/
     1                ,5X,' Dip of orebody            = ',f9.3,' (deg.)'/
     1                ,5X,' Rock mass modulus         = ',f9.3,' (MPa)'/
     1                ,5X,' Unit weight of rock       = ',f9.3,' (MN/cu.m.)'
```

```
     1/                ,5X,' Uniaxial compressive strength= ',f9.3,' (MPa)'/
     1                 ,5X,' Phi angle (friction)           = ',f9.3,' (deg.)')
     IF (INTY.EQ.1) THEN
     WRITE(*,6)
6    FORMAT('0 ***      PLANE STRAIN CONDITIONS                ***'/)
     ELSE
     WRITE(*,7)PR,AB
7    FORMAT('0 ***      RECTANGULAR PLATE CONDITIONS   ***'//
     1                 5X,' Poisson ratio                   = ',f9.3,/
     1                 5X,' A/B ratio                       = ',f9.3,//)
     ENDIF
C    CALL BEEPIT
     CORDAT='n'
     WRITE(*,*)' If data is correct, enter Y, if incorrect enter N.'
     READ (*,'(a1)')CORDAT
     IF ((CORDAT.EQ.'n').OR.(CORDAT.EQ.'N')) GOTO 2
```

```
c
c                          PRINT VERIFIED DATA
c
```

```
     NPR=2
     OPEN(NPR,FILE='LPT1')
     WRITE(NPR,8)
8    FORMAT('1       *** VOUSSOIR BEAM SOLUTION ***'/
     1          10X,'  ----------------------------------------------'/)
     WRITE(NPR,3)
     WRITE(NPR,4)DATE,SITE,PROJ,STOPE
     WRITE(NPR,5)S,T,Dip,E,Uniwt,UCS,PHI
     IF(INTY.EQ.1) THEN
     WRITE(NPR,6)
     ELSE
     WRITE(NPR,7)PR,AB
     ENDIF
```

```
c
c                          INITIALIZE
c
```

```
     DEGRAD=180./3.141592654
     DIPR=DIP/DEGRAD
     PHIR=PHI/DEGRAD
     EN=.1
     Z=T*(1.-(EN*2./3.))
     FC=1.E6
     IF (INTY.EQ.2)  GOTO 200
```

```
c
c                              PLANE STRAIN
c
10      FCMAX=FC
        FC=(UNIWT*COS(DIPR)*S**2.)/(4.*EN*Z)
        FAV=((2./3.)+(EN/2.))*FC/2.
        EL=S+(8.*Z**2./(3.*S))
        ELDEL=FAV*EL/E
        Z2=((8.*Z**2./(3.*S))-ELDEL)*3.*S/8.
        IF (Z2.LT.0.) THEN
        WRITE(*,*)' Buckling failure imminent!!!'
        WRITE(NPR,*)' Buckling failure imminent!!!'
        WRITE(NPR,*)
C       CALL BEEPIT
        GOTO 75
        ENDIF
        Z=SQRT(Z2)
        EN=(3./2.)*(1.-Z/T)
        IF (FC.LT.FCMAX) THEN
        FCL=FC
        ENL=EN
        Zl=Z
c       WRITE (NPR,20)EN,FC,Z,FCMAX
        GOTO 10
        ENDIF
c       WRITE (NPR,20)EN,FC,Z,FCMAX
20      FORMAT(5X,' N= ',F5.4,5X,' Fc= ',F8.4,5x,' Z = ',F8.4,5X,' Fcmax
        1= ',F8.4)
25      FOS1=(FCL*ENL/(UNIWT*S))*TAN(PHIR)
        FOS2=UCS/FCL

        WRITE(NPR,27)ENL,ZL,FCL
27      FORMAT('0       n = ',F4.2,5X,' Z = ',F4.2,' m. ',5X,' Fc = ',F5.2
        1,' MPa')
        WRITE(NPR,30)FOS1
30      FORMAT('0       FOS against abutment shear failure = ',f8.3)
        WRITE(NPR,40)FOS2
        WRITE(NPR,*)
40      FORMAT('0       FOS against compressive failure     = ',f8.3)
        WRITE(NPR,*)
75      INOTH=0
        WRITE(*,*)'TO DO ANOTHER ANALYSIS, ENTER 1'
        READ(*,'(I1)')INOTH
        IF (INOTH.EQ.1)  GOTO 2

        STOP
```

```
c
c                              RECTANGULAR PLATE
c
200     WRITE(*,*)'IS THE ROOF SPAN -S-, THE LONG OR SHORT SPAN?(L or S)'
        READ(*,'(A1)')SAB
        IF ((SAB.NE.'L').AND.(SAB.NE.'l').AND.(SAB.NE.'S').AND.(SAB.NE.'s'
        1)) GOTO 200
        IF ((SAB.EQ.'L').OR.(SAB.EQ.'l')) THEN
        B=S
        A=AB*B
        WRITE(NPR,*)'       Roof span is long span (b)'
        ELSE
        A=S
        B=A/AB
        WRITE(NPR,*)'       Roof span is short span (a)'
        ENDIF
210  Fcmax=Fc
   Y=(A/2.)*(SQRT(AB**2.+3.)-AB)
   Fc=(UNIWT*A**2.*(1./4.-(Y*AB)/(3.*A)))/(EN*Z)*COS(DIPR)
   IF (S.EQ.A)  FAV=.5*FC*((2./3.)+(EN/2.))
   IF (S.EQ.B)  FAV=(7./12.)*AB*FC
   EL=S+(8/3.)*(Z**2/S)
   ELDEL=FAV*EL*(1.-AB*PR)/E
   Z2=(3.*S/8.)*(8.*Z**2./(3.*S)-ELDEL)
   IF (Z2.LT.0.) THEN
    WRITE(*,*)' Buckling failure imminent!!!'
    WRITE(NPR,*)' Buckling failure imminent!!!'
    WRITE(NPR,*)
C     CALL BEEPIT
    GOTO 80
   ENDIF
   Z=SQRT(Z2)
   EN=(3./2.)*(1.-Z/T)
   IF (FC.LT.FCMAX) THEN
    FCL=FC
    ENL=EN
    ZL=Z

C    WRITE (NPR,20)EN,FC,Z,FCMAX

    GOTO 210
   ELSE

C    WRITE (NPR,20)EN,FC,Z,FCMAX

   FOS1=(FCL*ENl*B/(UNIWT*A*(B-Y)))*TAN(PHIR)
   FOS2=UCS/FCL
```

```
      WRITE(NPR,27)ENL,ZL,FCL
      WRITE(NPR,30)FOS1
      WRITE(NPR,40)FOS2
      WRITE(NPR,*)
     ENDIF
80    INOTH=0
      WRITE(*,*)' TO DO ANOTHER ANALYSIS, ENTER 1'
      READ(*,'(I1)')INOTH
      IF (INOTH.EQ.1) GOTO 2

      STOP
      END
```

APPENDIX 5-F

DOCUMENTATION AND PROGRAM LISTING FOR SUPPORT INTERACTION ANALYSIS (TUNNEL)

```
10    ' TUNNEL - Rock-support interaction analysis, E.Hoek, January 1985.
20    ' Reference:    Hoek. E and Brown, E.T. Underground Excavations in Rock,
30    '               The Institution of Mininig and Metallurgy, London, 1983,
40    ' pages 248 - 286.
50    '
60    ' Dimensioning of arrarys and definition of strings
70    '
80    DIM A(31): D$="i"
90    AA$ = "tunnel diameter"          :AB$ = "in situ stress"
100   AC$ = "intact rock u.c.s"        :AD$ = "m, original rock mass"
110   AE$ = "s, original rock mass"    :AF$ = "E, original rock mass"
120   AG$ = "Poisson ratio, rock"      :AH$ = "mr, broken rock mass"
130   AI$ = "sr, broken rock mass"     :AJ$ = "unit wt., broken rock"
140   AK$ = "Ec, concrete"             :AL$ = "Poisson ratio, concrete"
150   AM$ = "lining thickness"         :AN$ = "concrete u.c.strength"
160   AO$ = "set flange width"         :AP$ = "set section depth"
170   AQ$ = "cross-sectional area"     :AR$ = "moment of inertia"
180   AS$ = "Es, steel"                :AT$ = "yield strength, steel"
190   AU$ = "set spacing"              :AV$ = "angle between blocks"
200   AW$ = "block thickness"          :AX$ = "Eb, block material"
210   AY$ = "bolt length"              :AZ$ = "bolt diameter"
220   BA$ = "Es, bolt material"        :BB$ = "bolt constant Q"
230   BC$ = "bolt failure load"        :BD$ = "radial bolt spacing"
240   BE$ = "long. bolt spacing"       :BF$ = "tunnel data "
250   BG$ = "concrete or shotcrete lining"
260   BH$ = "steel set support"        :BI$ = "rockbolt support "
261   BJ$ = "support"                  :BK$ = "pressure"
262   BL$ = "roof"                     :BM$ = "sidewall"
263   BN$ = "floor"                    :BO$ = "displacement"
264   BP$ = "plastic zone"             :BQ$ = "thickness
270   '
280   ' Display of first page
290   '
300   CLS:KEY OFF:LOCATE 12,20:COLOR 0,7
310   PRINT "ROCK-SUPPORT INTERACTION ANALYSIS"
320   COLOR 7,0:LOCATE 16,25
330   INPUT "Tunnel diameter = ",A(1):A(1)=A(1)/2
340   GOTO 470
440   '
450   ' Display of second page
460   '
470   CLS:KEY OFF:PRINT "Analysis no.";C$
480   PRINT:COLOR 0,7:PRINT BF$;:COLOR 7,0
490   LOCATE 3,40:COLOR 0,7:PRINT BH$:COLOR 7,0
500   PRINT AA$;:          PRINT TAB(40) AO$
510   PRINT AB$;:          PRINT TAB(40) AP$
520   PRINT AC$;:          PRINT TAB(40) AQ$
530   PRINT AD$;:          PRINT TAB(40) AR$
540   PRINT AE$;:          PRINT TAB(40) AS$
550   PRINT AF$;:          PRINT TAB(40) AT$
560   PRINT AG$;:          PRINT TAB(40) AU$
```

```
570   PRINT AH$;:          PRINT TAB(40) AV$
580   PRINT AI$;:          PRINT TAB(40) AW$
590   PRINT AJ$;:          PRINT TAB(40) AX$
600   PRINT:COLOR 0,7:PRINT BI$ ;: COLOR 7,0
610   LOCATE 15,40:COLOR 0,7: PRINT BG$ : COLOR 7,0
620   PRINT AY$;:          PRINT TAB(40) AK$
630   PRINT AZ$;:          PRINT TAB(40) AL$
640   PRINT BA$;:          PRINT TAB(40) AM$
650   PRINT BB$;:          PRINT TAB(40) AN$
660   PRINT BC$: PRINT BD$
670   PRINT BE$: PRINT:GOSUB 930:GOTO 3030
680   '
690   ' definition of soft keys
700   '
710   FOR I = 1 TO 10:KEY I,"":NEXT I
720   KEY 1,"a":KEY 2,"b":KEY 3,"c":KEY 4,"d"
730   KEY 5,"e":KEY 6,"f":KEY 7,"g"
740   '
750   ' display of soft key options
760   '
761   LOCATE 25,1 : PRINT "1";
762   LOCATE 25,3 : COLOR 0,7 :PRINT "input";: COLOR 7,0
770   LOCATE 25,10: PRINT "2";
780   LOCATE 25,12: COLOR 0,7 :PRINT "calculate";: COLOR 7,0
790   LOCATE 25,22: PRINT "3";
800   LOCATE 25,24: COLOR 0,7 :PRINT "quit";: COLOR 7,0
881   RETURN
890   '
900   ' Subroutine for entry and display of title
910   '
930   IF D$="r" THEN 970
940   LOCATE 1,1:COLOR 0,7:PRINT "Analysis no. ";:COLOR 2,0
960   LOCATE 1,14:LINE INPUT "",C$
970   LOCATE 1,1:PRINT "Analysis no. ";
980   LOCATE 1,14:PRINT C$;
990   P=1:Q=1:Y$="Analysis no. ":RETURN
1010  '
1020  ' Entry and display of data
1030  '
1040  J=1:  D=1:  A=4  :B=30:X$=AA$:GOSUB 1500:RETURN     'tunnel radius
1050  J=2:  D=1:  A=5  :B=30:X$=AB$:GOSUB 1500:RETURN     'in situ stresss
1060  J=3:  D=1:  A=6  :B=30:X$=AC$:GOSUB 1500:RETURN     'intact rock ucs
1070  J=4:  D=1:  A=7  :B=30:X$=AD$:GOSUB 1500:RETURN     'm,orignial rock
1080  J=5:  D=1:  A=8  :B=30:X$=AE$:GOSUB 1500:RETURN     's,original rock
1090  J=6:  D=1:  A=9  :B=30:X$=AF$:GOSUB 1500:RETURN     'E,original rock
1100  J=7:  D=1:  A=10:B=30:X$=AG$:GOSUB 1500:RETURN     'Poisson ratio
1110  J=8:  D=1:  A=11:B=30:X$=AH$:GOSUB 1500:RETURN     'mr, broken rock
1120  J=9:  D=1:  A=12:B=30:X$=AI$:GOSUB 1500:RETURN     'sr, broken rock
1130  J=10:D=1:  A=13:B=30:X$=AJ$:GOSUB 1500:RETURN     'unit wt., rock
1140  J=11:D=1:  A=16:B=30:X$=AY$:GOSUB 1500:RETURN     'rockbolt length
1150  J=12:D=1:  A=17:B=30:X$=AZ$:GOSUB 1500:RETURN     'bolt diameter
```

```
1160  J=13:D=1  :A=18:B=30:X$=BA$:GOSUB 1500:RETURN     'Es, bolt matl.
1170  J=14:D=1  :A=19:B=30:X$=BB$:GOSUB 1500:RETURN     'constant Q
1180  J=15:D=1  :A=20:B=30:X$=BC$:GOSUB 1500:RETURN     'failure load
1190  J=16:D=1  :A=21:B=30:X$=BD$:GOSUB 1500:RETURN     'radial spacing
1200  J=17:D=1  :A=22:B=30:X$=BE$:GOSUB 1500:RETURN     'long. spacing
1210  J=18:D=40:A=4  :B=70:X$=AO$:GOSUB 1500:RETURN     'flange width
1220  J=19:D=40:A=5  :B=70:X$=AP$:GOSUB 1500:RETURN     'section depth
1230  J=20:D=40:A=6  :B=70:X$=AQ$:GOSUB 1500:RETURN     'x-sect. area
1240  J=21:D=40:A=7  :B=70:X$=AR$:GOSUB 1500:RETURN     'moment inertia
1250  J=22:D=40:A=8  :B=70:X$=AS$:GOSUB 1500:RETURN     'Es, steel
1260  J=23:D=40:A=9  :B=70:X$=AT$:GOSUB 1500:RETURN     'yield strength
1270  J=24:D=40:A=10:B=70:X$=AU$:GOSUB 1500:RETURN     'set spacing
1280  J=25:D=40:A=11:B=70:X$=AV$:GOSUB 1500:RETURN     'blocking angle
1290  J=26:D=40:A=12:B=70:X$=AW$:GOSUB 1500:RETURN     'block thickness
1300  J=27:D=40:A=13:B=70:X$=AX$:GOSUB 1500:RETURN     'Eb, block matl.
1310  J=28:D=40:A=16:B=70:X$=AK$:GOSUB 1500:RETURN     'Ec, concrete
1320  J=29:D=40:A=17:B=70:X$=AL$:GOSUB 1500:RETURN     'Poisson's ratio
1330  J=30:D=40:A=18:B=70:X$=AM$:GOSUB 1500:RETURN     'lining thiickness
1340  J=31:D=40:A=19:B=70:X$=AN$:GOSUB 1500:RETURN     'concrete u.c.s.
1500  '
1510  'Subroutine for data entry and display
1520  '
1530  IF D$="r" THEN 1595
1550  LOCATE A,D:COLOR 0,7:PRINT X$:COLOR 2,0
1570  IF D$="i" THEN LOCATE A,B-4:PRINT "          ";
1580  LOCATE A,B:INPUT "",A(J)
1590  LOCATE A,B:PRINT "          "
1595  IF A(J)>9999 OR A(J)<.001 THEN 1620
1610  B=B-2:LOCATE A,B:PRINT USING "####.###";A(J):GOTO 1630
1620  B=B-3:LOCATE A,B:PRINT USING "##.##^^^^";A(J)
1630  LOCATE A,D:PRINT X$
1640  P=A:Q=D:Y$=X$:RETURN
2000  '
2010  'data entry subroutine - tunnel
2020  '
2030  GOSUB 1040 :GOSUB 1050 :GOSUB 1060 :GOSUB 1070
2040  GOSUB 1080 :GOSUB 1090 :GOSUB 1100 :GOSUB 1110
2050  GOSUB 1120 :GOSUB 1130 :A(1)=A(1)/2:RETURN
2051  '
2052  ' data entry subroutine - rockbolts
2053  '
2060  GOSUB 1140 :GOSUB 1150 :GOSUB 1160 :GOSUB 1170
2070  GOSUB 1180 :GOSUB 1190 :GOSUB 1200 :RETURN
2071  '
2072  ' data entry subroutine - steel sets
2073  '
2074  GOSUB 1210 :GOSUB 1220 :GOSUB 1230 :GOSUB 1240
2080  GOSUB 1250 :GOSUB 1260 :GOSUB 1270 :GOSUB 1280
2090  GOSUB 1290 :GOSUB 1300 :RETURN
2091  '
2092  ' data entry subroutine - concrete/shotcrete lining
```

```
2093  '
2100  GOSUB 1310 :GOSUB 1320 :GOSUB 1330 :GOSUB 1340
2101  RETURN
2500  '
2510  ' Scan keyboard for soft-key input
2520  '
2530  Q$ = INKEY$ : IF Q$ = "" THEN 2530
2540  IF LEN(Q$)=2 THEN Q$=RIGHT$(Q$,1)
2550  IF Q$ = "a" THEN 3000       'input
2590  IF Q$ = "b" THEN 5000       'calculate
2610  IF Q$ = "c" THEN END ELSE 2530 'quit
3000  '
3010  'data entry selection
3020  '
3030  KEY ON:KEY OFF:LOCATE 25,1:PRINT "input selection:";
3040  LOCATE 25,19: PRINT "t";
3050  LOCATE 25,21:COLOR 0,7:PRINT "tunnel";: COLOR 2,0
3060  LOCATE 25,30 : PRINT "r";
3070  LOCATE 25,32:COLOR 0,7:PRINT "rockbolts";:COLOR 2,0
3080  LOCATE 25,44:PRINT "s";
3090  LOCATE 25,46:COLOR 0,7:PRINT "steel sets";:COLOR 2,0
3100  LOCATE 25,60:PRINT "c";
3110  LOCATE 25,62:COLOR 0,7:PRINT "concrete lining";:COLOR 2,0
3120  Q$=INKEY$ :IF Q$="" THEN 3120
3130  IF Q$="t" OR Q$="T" THEN 3200
3140  IF Q$="r" OR Q$="R" THEN 3250
3150  IF Q$="s" OR Q$="S" THEN 3300
3160  IF Q$="c" OR Q$="C" THEN 3350 ELSE 3120
3200  G=1:GOSUB 2030:KEY ON:KEY OFF:GOSUB 710:GOTO 2500
3250  G=2:GOSUB 2060:KEY ON:KEY OFF:GOSUB 710:GOTO 2500
3300  G=3:GOSUB 2074:KEY ON:KEY OFF:GOSUB 710:GOTO 2500
3350  G=4:GOSUB 2100:KEY ON:KEY OFF:GOSUB 710:GOTO 2500
5000  IF G=1 THEN CLS:GOTO 5019
5011  IF G=2 THEN 7000
5012  IF G=3 THEN 7000
5013  IF G=4 THEN 7000 ELSE 2530
5019  LOCATE 1,15:COLOR 0,7:PRINT "Calculation of required  ";
5020  PRINT "support lines for tunnel ": COLOR 2,0
5021  LOCATE 3,4 :PRINT BM$;:LOCATE 4,4 :PRINT BO$;
5022  LOCATE 3,20:PRINT "--------";:LOCATE 3,29:PRINT BJ$;
5023  LOCATE 3,37:PRINT BK$:LOCATE 3,46:PRINT "-------"
5024  LOCATE 4,20:PRINT BL$:LOCATE 4,33:PRINT BM$;:LOCATE 4,48
5025  PRINT BN$:LOCATE 3,59:PRINT BP$;:LOCATE 4,59:PRINT BQ$
5030  GOSUB 6000:SPS=0:GOSUB 6040:Y=6:GOSUB 5500
5040  LOCATE Y,33:COLOR 0,7:PRINT"       ":COLOR 7,0
5050  LOCATE Y,33:INPUT "",PRES$
5060  IF LEN(PRES$)=0 THEN LOCATE Y,33:PRINT "       ":GOTO 5400
5070  SPS=VAL(PRES$):GOSUB 6040:GOSUB 5500:GOTO 5040
5400  Q$=INKEY$:IF Q$="" THEN 5400
5410  GOTO 470
5497  '
```

```
5498  ' Subroutine for display of data
5499  '
5500  LOCATE Y,4:PRINT USING "##.####";UI
5510  LOCATE Y,20:PRINT USING "####.##";SPR
5520  LOCATE Y,33:PRINT USING "####.##";SPS
5530  LOCATE Y,48:PRINT USING "####.##";SPF
5535  Z=A(1)*(RERI-1):IF FLAG=1 THEN Z=0
5540  LOCATE Y,59:PRINT USING "####.##";Z
5550  Y=Y+1
5560  IF Y=23 THEN Y=6:RETURN
5987  '
5988  ' Subroutine for tunnel support
5999  '
6000  M=.5*SQR((A(4)/4)^2+A(4)*A(2)/A(3)+A(5))-A(4)/8
6010  SIGRE=A(2)-M*A(3):RETURN
6040  FLAG=O:IF SPS<SIGRE GOTO 6500
6050  UI=A(1)*(1+A(7))*(A(2)-SPS)/A(6)
6060  SPR=SPS:SPF=SPS:FLAG=1:RETURN
6500  D=-A(4)/(A(4)+4*SQR(A(4)*SIGRE/A(3)+A(5)))
6501  N=2*SQR(SIGRE/(A(8)*A(3))+A(9)/A(8)^2)
6502  UERE=(1+A(7))*M*A(3)/A(6)
6510  RERI=EXP(N-2*SQR(SPS/(A(8)*A(3))+A(9)/A(8)^2))
6520  IF RERI<=SQR(3) THEN R=2*D*LOG(RERI) ELSE R=1.1*D
6530  EAV=2*UERE*RERI^2/((RERI^2-1)*(1+1/R))
6540  A=(2*UERE-EAV)*RERI^2
6550  UI=A(1)*(1-SQR((1-EAV)/(1+A)))
6560  SPR=SPS+A(10)*A(1)*(RERI-1)
6570  SPF=SPS-A(10)*A(1)*(RERI-1):RETURN
7000  LOCATE 21,40:PRINT "Initial deformation = "
7010  LOCATE 22,40:PRINT "Final deformation  = "
7020  LOCATE 23,40:PRINT "Max support press = "
7030  LOCATE 21,62:COLOR 0,7:PRINT "        ":COLOR 7,0
7035  LOCATE 21,62:INPUT "",UIO
7040  LOCATE 21,62:PRINT USING "####.####";UIO
7050  IF G=2 THEN GOSUB 7500:GOTO 7075
7060  IF G=3 THEN GOSUB 8000:GOTO 7075
7070  IF G=4 THEN GOSUB 8500:GOTO 7075 ELSE 3120
7075  UFINAL=UIO+PSMAX*A(1)*STIF
7080  LOCATE 22,62:PRINT USING "####.####";UFINAL
7090  LOCATE 23,62:PRINT USING "####.####";PSMAX
7100  GOTO 2530
7497  '
7498  ' Subroutine for rockbolts
7499  '
7500  STIF=A(16)*A(17)/A(1)
7510  STIF=STIF*(4*A(11)/(3.14159*A(12)^2*A(13))+A(14))
7520  PSMAX=A(15)/(A(16)*A(17)):RETURN
7987  '
7988  ' Subroutine for steel sets
7999  '
8000  ANG=.5*A(25)*3.141593/180
```

```
8010  STIF=ANG*(ANG+SIN(ANG)*COS(ANG))/(2*(SIN(ANG))^2)-1
8020  STIF=STIF*A(24)*A(1)^3/(A(22)*A(21))
8030  STIF=STIF+A(24)*A(1)/(A(22)*A(20))
8040  STIF=STIF+2*A(24)*ANG*A(26)/(A(27)*A(18)^2)
8045  TEMP=3*A(21)+A(19)*A(20)*(A(1)-(A(26)+A(19)/2))*(1-COS(ANG))
8050  PSMAX=3*A(20)*A(21)*A(23)/(2*A(24)*A(1)*ANG)
8060  PSMAX=PSMAX/TEMP
8070  RETURN
8487  '
8488  ' Subroutine for shotcrete or concrete
8499  '
8500  STIF=A(1)^2*(1-2*A(29))+(A(1)-A(30))^2*(1+A(29))
8510  STIF=STIF/(A(28)*(A(1)^2-(A(1)-A(30))^2))
8520  PSMAX=.5*A(31)*(1-(A(1)-A(30))^2/A(1)^2)
8530  RETURN
```